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Idaho State Highway 55, North Fork of the Payette River near Smiths Ferry. (Photo courtesy of the Idaho Department of Highways.)

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Bridge over Loire River, Bloix, France.

Status of Concrete-Polymer Composites in the United States and Abroad

OFFICE OF DEVELOPMENT

Reported by ANTHONY M. LIZZIO,
Structural Engineer,
Implementation Division

Introduction

DURING the 19th century, civil/structural engineers saw a dramatic dawn with the invention of the Bessemer and Open-Hearth processes for the manufacture of steel which could be used either alone or added to concrete to produce reinforced concrete. These two materials created a tremendous surge in industrial history and have formed the basis of construction engineering during this century. However, since these inventions by Sir Henry Bessemer and Fredrick Siemens in 1856, (1) [1] there has been little change in basic construction materials. The most noteworthy improvement has been an increase in the permissible design stresses for steel and concrete due to greater confidence in the materials and improved quality control. These factors combined with the increasing use of high tensile steels have led to more refined designs and greater elegance in structures.

Only recently has it appeared possible that man's inventiveness will produce the equivalent of an epoch-making structural material such as steel. That product may well be a composite of plastic or *polymer* and concrete. The fact that there is a material of such great potential should be an irresistible challenge to the forward thinking civil/structural engineer.

Definitions

The term polymer and other technical terms relating to polymer chemistry used in this article are defined in the following statements:

Polymer.—Polymer is the technical name given by the plastics industry to natural and synthetic resin plastics which are organic compounds of high molecular weight. Chemically, the basic resins consist of combinations of carbon with hydrogen, oxygen, nitrogen, chlorine, fluorine, silicon and other elements.

Monomer.—The starting material from which a polymer is formed is called a monomer. It is the simple unpolymerized form of a chemical compound having a

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[1] Italic numbers in parentheses identify the references listed on page 135.

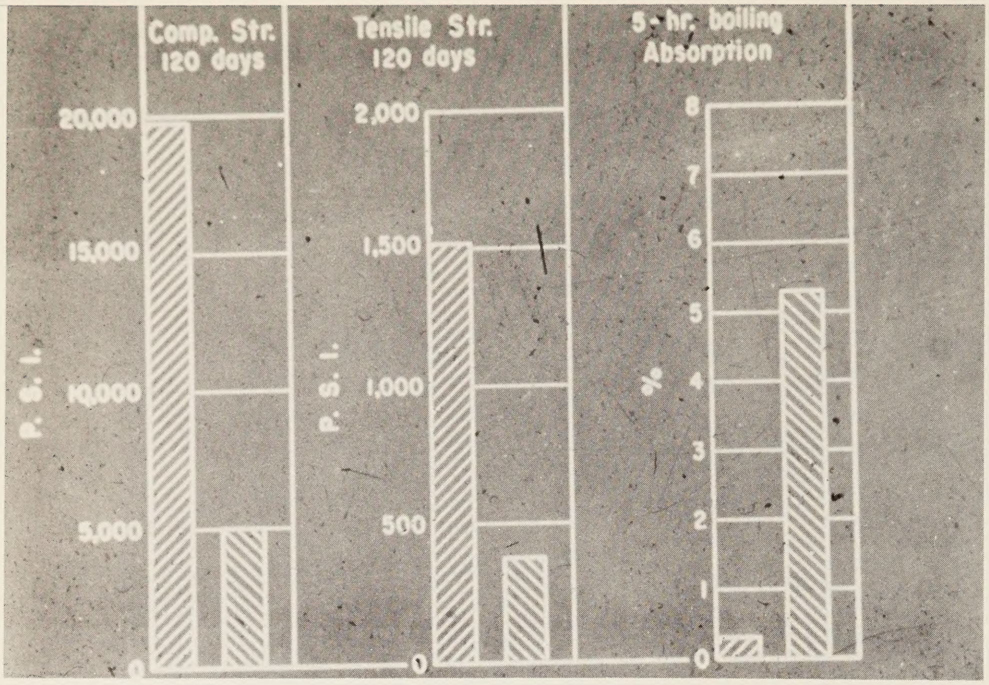

Figure 1.—Properties of concrete with full impregnation (left) and without.

low molecular weight; for example, styrene and methyl methacrylate.

Polymerization.—A chemical reaction in which monomers are united to form polymers. The smaller molecules combine in a chain reaction to form larger molecules that contain units of the original molecule.

Concrete and Plastic Composites

Composites of concrete, mortars, and various polymers or plastics are being studied intensively, both in the United States and abroad. The upsurge of interest in these materials can be attributed to the remarkable improvements which were obtained when the monomers, methyl methacrylate and styrene were added to conventional concrete and mortars and polymerized to form entirely new products. These products have increased strength, durability, abrasion resistance, resistance to chemical attack, and have reduced permeability, absorption and creep. The first specimens of what is now called polymer impregnated concrete were produced at the Brookhaven National Laboratory in Upton, N.Y., in late 1965, and the first laboratory evaluation and tests were conducted by the U.S. Bureau of Reclamation in Denver, Colo., in the early part of 1966 under sponsorship with the U.S. Atomic Energy Commission (*2*). Figure 1 shows compressive strength, tensile strength, and water absorption of these specimens compared to the values obtained from tests on similar specimens of conventional concrete. Figure 2 shows comparative results obtained from freeze-thaw tests and figure 3 shows the resistance of polymer impregnated concrete to chemical attack by 15 percent hydrochloric acid. These significantly optimistic laboratory results led to further investigations and the development of four distinct types of polymer composites which are defined in this article as follows:

(1) *Polymer impregnated concrete (PIC).*—A hardened portland cement concrete which is impregnated with a low viscosity monomer that is polymerized in situ.

(2) *Polymer cement concrete (PCC).*—A mixture of portland cement, water, aggregates and a liquid monomer. After the mixture has been placed, the monomer is polymerized.

(3) *Polymer concrete (PC).*—A mixture of liquid monomer and aggregate. After placement, the mixture is polymerized.

(4) *Polymer mortar (PM).*—A mixture of liquid monomer and fine aggregate. It is polymerized after placement.

Since 1965, most of the research efforts in the United States have centered on the PIC, primarily because it was the first composite and there is substantial evidence that its physical properties are superior to those of conventional concrete. One of the first research reports (*3*) showed that for fully impregnated precast concrete specimens—

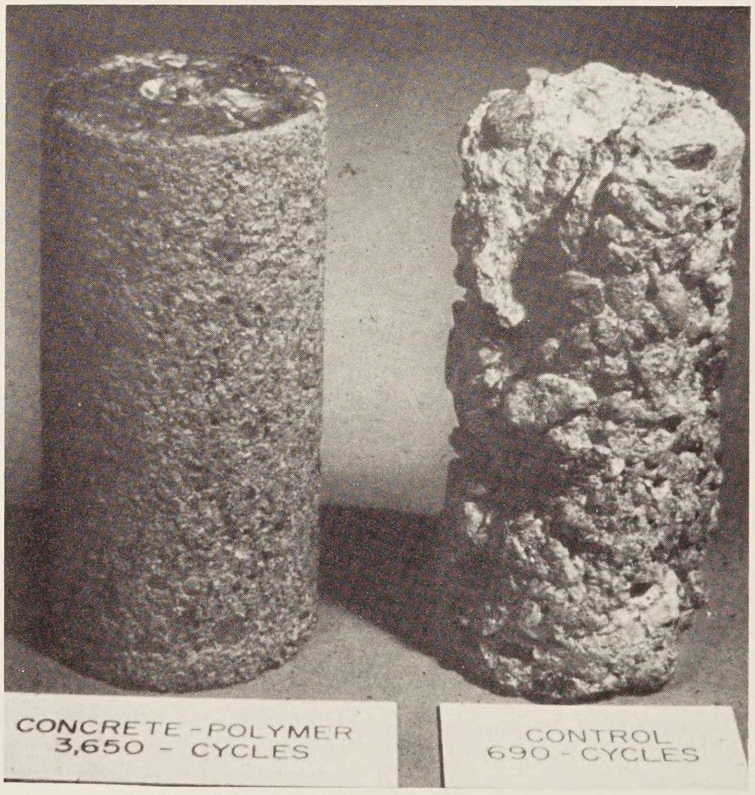

Figure 2.—Freeze thaw resistance.

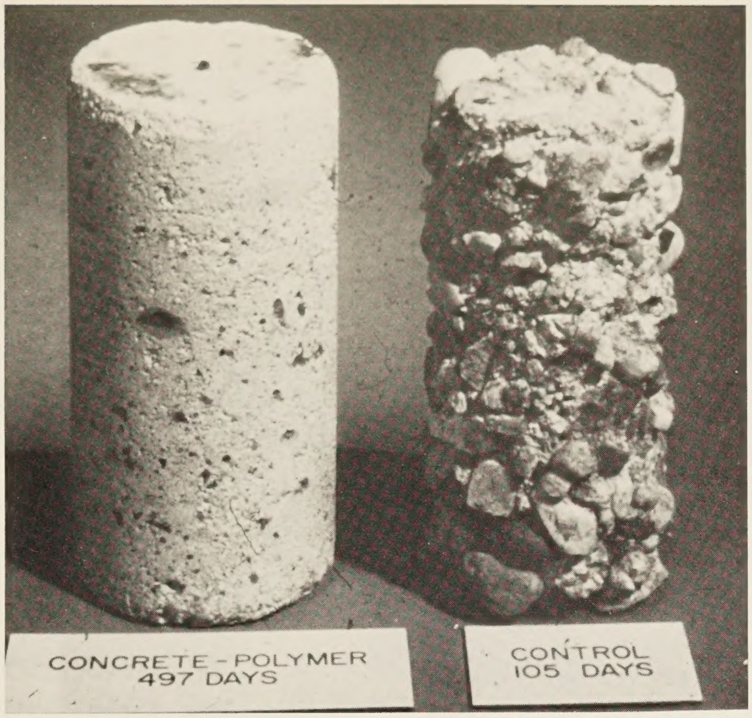

Figure 3.—Resistance to 15 percent hydrochloric acid.

- Compressive strength of PIC is four times that of a conventional concrete.
- Tensile strength increases almost four times.
- Modulus of elasticity increases almost two times.
- Modulus of rupture increases almost four times.
- Flexural modulus of elasticity increases almost 1.5 times.
- Resistance to freezing and thawing is five times greater.
- Hardness-impact values (L hammer) increase more than 1.7 times.
- Water permeability becomes negligible.
- Water absorption is greatly reduced.
- Corrosion due to distilled water and sulfate brines is negligible.

These exciting results were obtained with polymer concentrations or loadings of approximately 6–8 percent, by weight, of the dry, hardened concrete.

The substantial advantages of PIC have encouraged researchers to simplify the impregnation process, improve quality and reduce costs (*4*). For example, the initial process required a radiation source to polymerize the liquid monomer. Subsequently, it was found that chemical catalysts, accelerators and a mild heat would provide a more practical procedure while producing similar results at a lower cost. And researchers currently believe that an even more economical thermal-catalytic process for polymerization can be perfected.

In its simplest form, the process of producing fully impregnated polymer concrete requires the following six steps (*5*):

(1) Using a conventional portland cement concrete to cast a concrete beam, slab, pipe, or other product.

(2) Curing the product.

(3) Drying the product.

(4) Soaking the dried product in a liquid monomer in a tank which can be partially evacuated and pressurized.

(5) Sealing the monomer impregnated product in a container or placing it in a warm water bath to preclude evaporation.

(6) Polymerizing the impregnated product, using a radiation or thermal-catalytic process.

A notable disadvantage of the PIC process is that individual products are limited by the size of the tank which is used for impregnation. Figure 4 shows the tank at Upton, N.Y., which can accommodate an article approximately 5 feet in diameter and 12 feet in length.

With respect to polymer cement concrete, studies have been underway to develop techniques for integrally mixing monomers with fresh concrete. However, to date little success has been obtained with PCC (*6*). The basic problem with PCC appears to be the incompatibility of most plastics with the water which is needed to hydrolize portland cement. Since PCC is a premixed material which has greater potential for field application than PIC, the research is continuing.

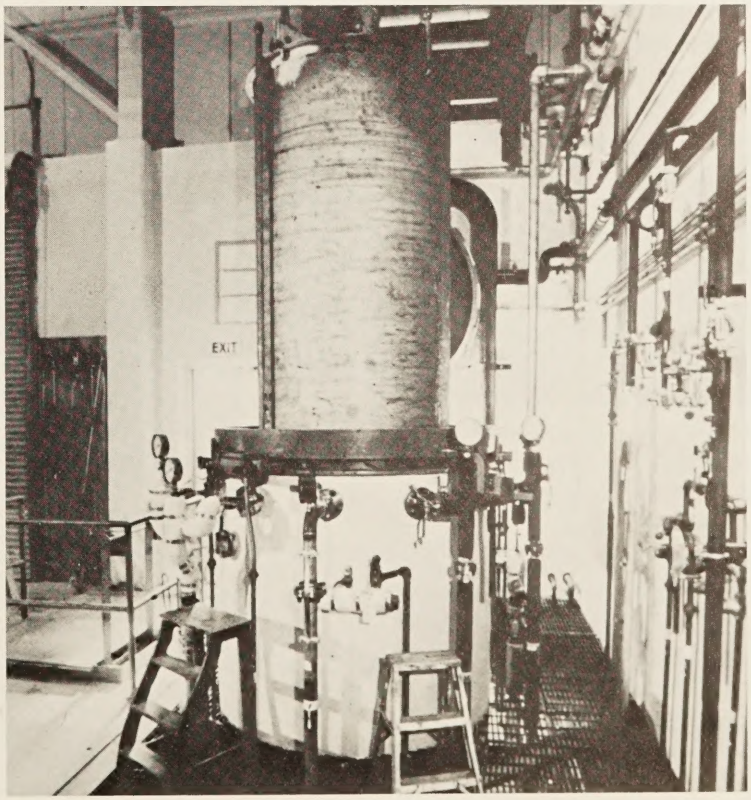

Figure 4.—Impregnation tank, Upton, N.Y.

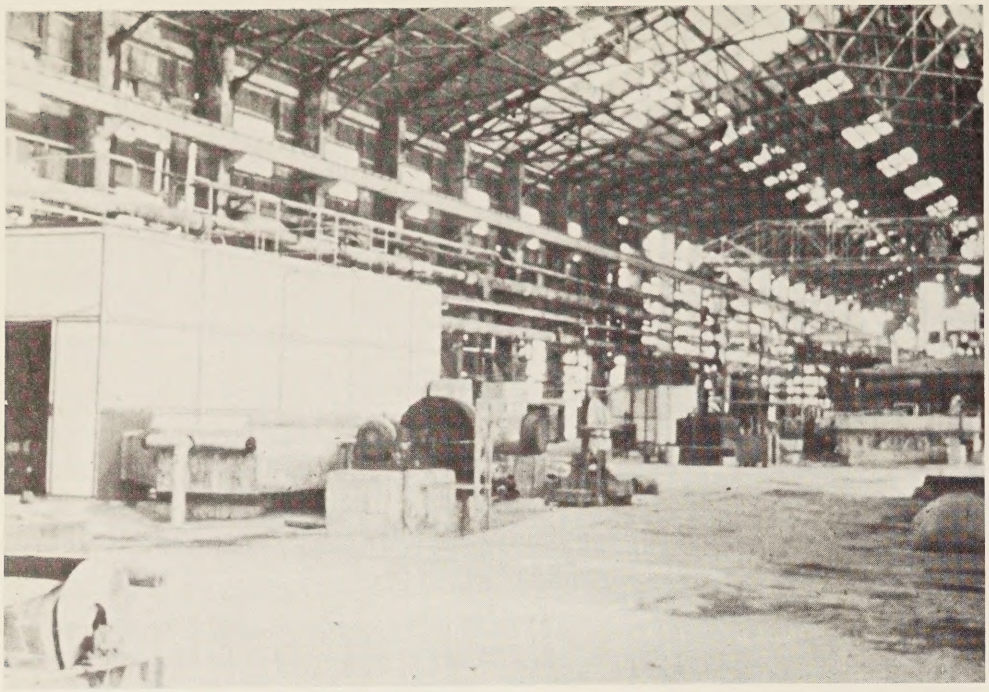

Figure 5.—PIC pilot plant, Kyushu, Japan.

Polymer concrete is also a premixed material but, since the mixture consists of only aggregates and a liquid monomer, the problems associated with incompatibility of systems that contain water and organic monomers are not present. Therefore, research in this area is being accelerated with efforts centered on the determination of optimum gradation and polymer loading to fill void spaces, and improved techniques for mixing, placing and polymerizing the materials. These investigations have not yet developed a product as desirable as PIC, but some recent tests indicate that PC might be the most promising of the new composites. However, it must be emphasized that investigators are still exploring the chemical and physical properties of polymer concretes and that a definite answer must await the completion of these studies.

Of great significance in the PC studies is the aggregate size and gradation. Studies are currently centering on devising means of reducing the specimen void volume and lowering the monomer requirements by varying proportions and aggregate size. The aggregate can be similar to that used in the conventional portland cement concrete, but other types of aggregate and filler including portland cement are being investigated.

Polymer mortar (PM) is a material similar to PC but does not have any coarse aggregate. Although PM is not under intensive study in the United States, there are some interesting potential applications such as the use of the material in roadway and bridge deck overlays, thick coatings to protect structures against chemical and mechanical attack, linings for hydraulic structures to prevent cavitation, and as a grouting material for prestressing cables and foundation wall anchor tie-backs.

Recent Developments in the United States

In the United States considerable research and development is already underway. A joint industry, U.S. Bureau of Reclamation, U.S. Federal Highway Administration (FHWA) project to design, fabricate, impregnate, and test a distinctive type of polymer impregnated precast slabs for use as bridge decking was initiated in June 1972. The deck panels will be 16 feet long and 4 feet wide, and a special impregnation tank will be fabricated to accommodate them. Such a system of bridge decking could be installed rapidly, not only on new structures, but also on old structures which have badly deteriorated decks that must be replaced.

The American Concrete Pipe Association, U.S. Bureau of Reclamation, and Brookhaven National Laboratory are cooperating in a research program to fabricate polymer impregnated concrete sewer and pressure pipes, without reinforcing steel, to determine strengths and performance under corrosive environments as compared with standard piping (5).

For the past 5 years, the Navy Department's Civil Engineering Laboratory has been studying the behavior of conventional concrete structures for underwater applications. Recently, this research was extended to polymer impregnated concrete structures because of PIC's high strength, durability, and low permeability, all of which are highly desirable when a material is used in underwater structures (7).

The Port of New York Authority pioneered in the development of a mechanized laydown technique for a thin PC overlay material for bridge decks and is

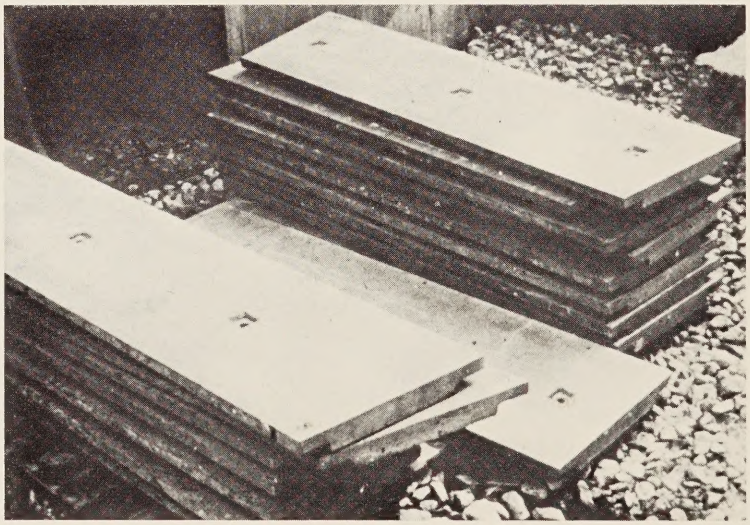

Figure 6.—PIC conduit covers.

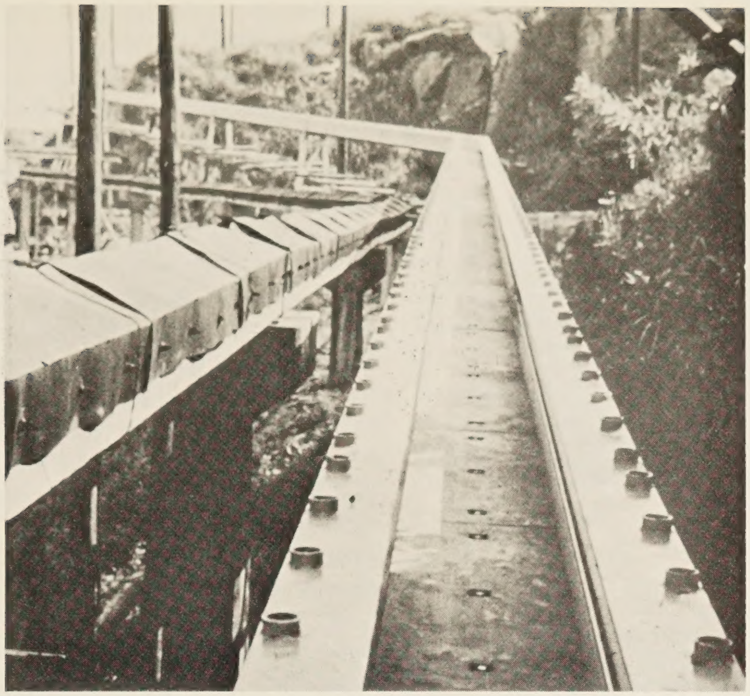

Figure 7.—High voltage electrical conduit, Shikoku, Japan.

in the process of evaluating the performance of its first major laydown (*8*).

The U.S. Bureau of Reclamation and the U.S. Bureau of Mines are interested in the use of polymer impregnation to strengthen mine support systems (*9*).

The U.S. Bureau of Reclamation is investigating tunnel supports and liners using a PIC system.

The Department of the Army has been experimenting in the use of polymer impregnated lightweight concrete for military housing applications (*10*).

The Office of Saline Water is interested in the potential use of corrosive resistant PIC for constructing distillation plants used in converting sea water to fresh water (*11*).

And of major importance to the highway engineering field is the effort underway by the FHWA Office of Research to determine the feasibility of partially impregnating a new bridge deck in order to fill the voids and keep deicing chemicals from penetrating. With respect to older decks which have deteriorated, polymerized monomer can supplant failed cement binder and renew the deck strength which will prolong the bridge life. Recent field tests indicate that the technique is feasible but a practical application method will require at least 2 more years for development.

Developments Abroad

Since issuance of a patent on March 2, 1971, to Brookhaven National Laboratory for the PIC process (*12*), engineers in several foreign countries have become well acquainted with the pioneer work conducted by the Brookhaven Laboratory and the Bureau of Reclamation. Already they are using this information to develop a variety of products.

The Japanese are offering various polymer impregnated products on the market including prestressed concrete structural beams, curtain walls, water and sewer pipe, high strength piling, and even such novelty items as figurines (*13*). Figure 5 shows a polymer impregnation plant in Omuta, Kyushu, Japan. The overhead bridge crane and the relatively large size impregnation tank can be seen in the background.

This plant fabricated a large number of PIC panels which were used to cover and protect a conduit for high voltage electrical cables. The panels shown in figure 6 are 300 mm. by 1,500 mm. by 30 mm. (about 12 in. by 59 in. by 1½ in.). In this case, it was important to eliminate steel reinforcing to avoid heat generation and potential power loss due to the ever present magnetic field around the cables. Similar PIC panels were successfully used as bottom cover plates on a viaduct which crossed a highly corrosive waste gas area. It was reported that PIC paneling was less expensive than that of an alternative solution using stainless steel panels. A similar conduit under construction at Shikoku, Japan, is shown in figure 7.

Another interesting application reported by the Japanese is the road heating panel shown in figure 8. This test panel is used to melt snow on a roadway.

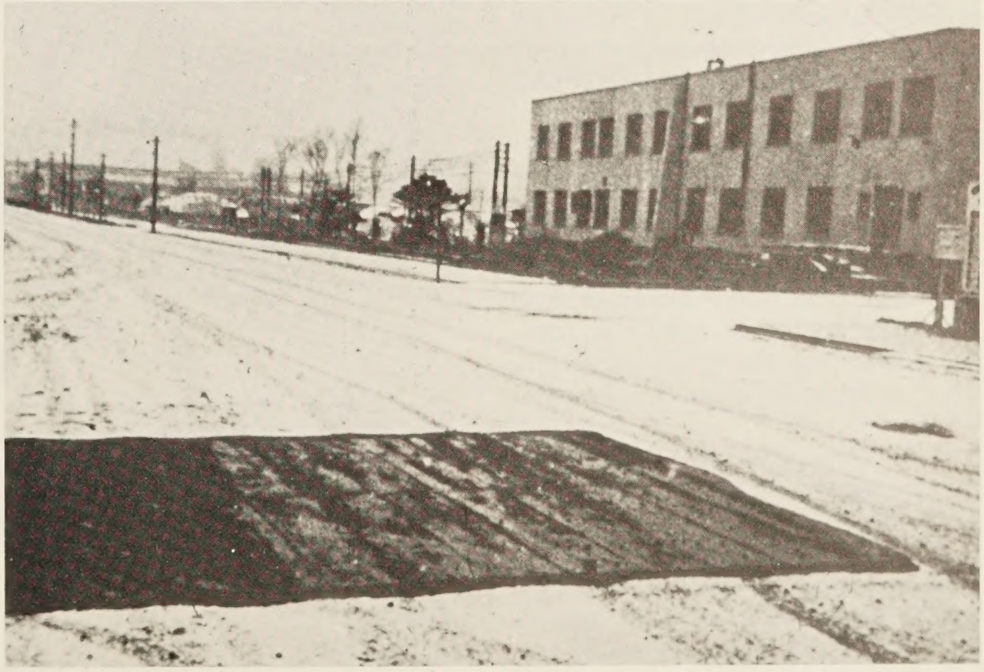

Figure 8.—Experimental PIC heating panel.

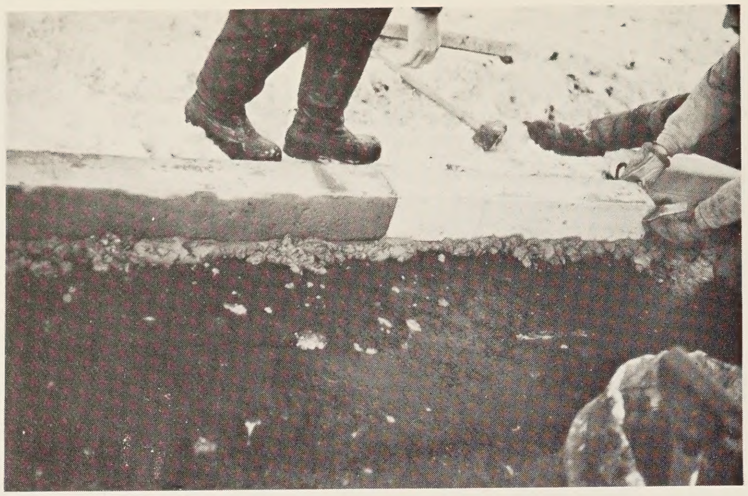

Figure 9.—PIC curbstone, Oslo, Norway.

Because of its negligible permeability and high strength, PIC is preferable to conventional concrete. Moreover, PIC is a better conductor of heat than is asphalt.

The Italians have developed polymer impregnation techniques that strengthen specimens up to 38,000 pounds per square inch using the PIC process and they are restoring priceless works of art with these techniques.

The Norwegians have placed 200 meters of curbstone made by a relatively simple partially impregnated concrete technique. Figure 9 shows the curbstone being installed on an Oslo, Norway, street (*14*).

A South African firm has built the pilot plant shown in figure 10. They plan to impregnate preformed concrete products such as wash basins, sinks, and bathtubs to supply the large underdeveloped South African market (*15*).

The Amsterdam Department of Public Works installed a polymer concrete sublayer in a new bascule bridge over the Ooster DokDoorgang Canal. The composite material was used instead of the traditional balsa wood blocks or steel framework to reduce installation costs and preclude risk of corrosion on an area where maintenance would have been impractical.

In the Federal Republic of Germany, the first experiments with plastics in surfacing were carried out in 1962. The first tests involved a mortar made with a plastic known as Vestopal. It was originally developed as a repair material for patching damaged concrete surfaces. Now, large areas that have suffered structural damage from repetitive heavy loads and extensive deicing salts are being resurfaced with polymer mortars made with various types of plastic, including Vestopal (*16*).

The Russians have indicated that polymer mortars and polymer concretes are being used increasingly in mining construction, buildings, bridges, and other structures including motorways. Their research efforts in this area can be traced back to 1957 (*17*), and are apparently quite extensive. Many types of monomers and variations of the same monomer have been studied to determine physical limitations and ranges for the polymer under investigation.

In France, a unique cantilevered bridge system has been developed. The system uses polymer mortar as a structural adhesive for gluing together prestressed concrete box girders; not as a last minute alternative to other methods of fixing, but as an integral part of the structural design. The system enables all work to be carried out from above, thus eliminating the need for falsework. This was particularly important in the construction of the Oleron Viaduct where tidal conditions made it difficult to bring materials to the piers. The use of conventionally packed mortar joints between the post tensioned precast units would have caused delays as such joints need to be rigidly supported until they are sufficiently strong to be prestressed which may take up to 7 days. Instead, a fast-curing polymer mortar that hardened within hours was used which, together with a reduced joint width, allowed each unit to be pre-

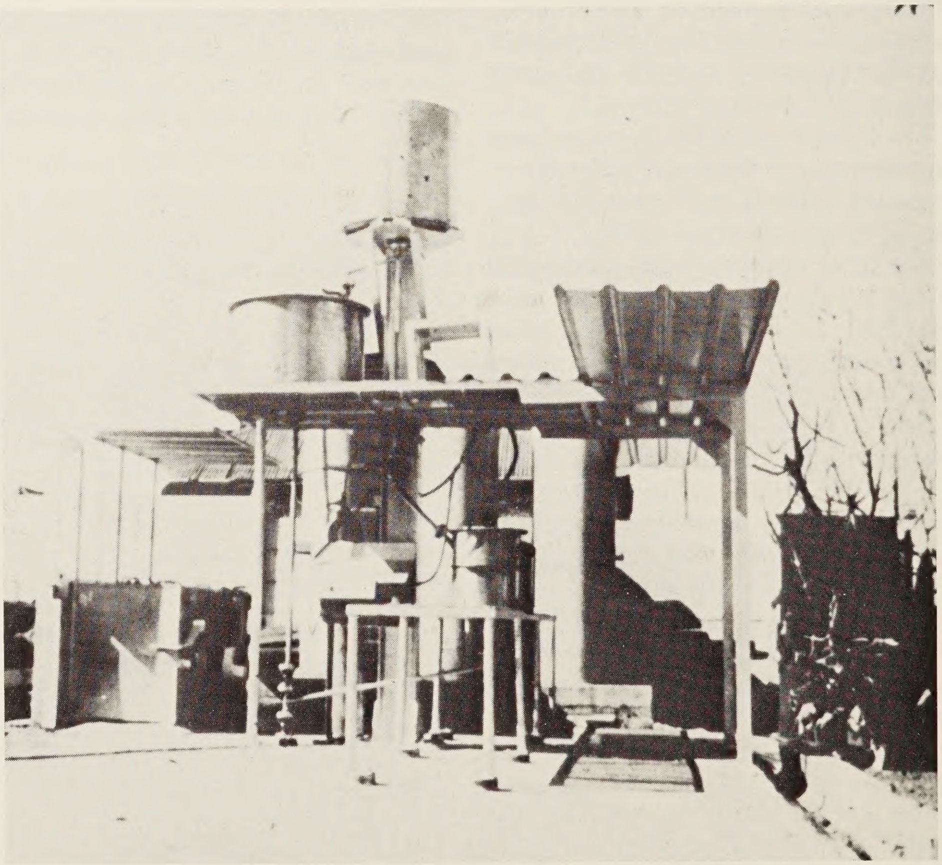

Figure 10.—PIC pilot plant, South Africa.

stressed to the bridge almost immediately after the joint was assembled. The 9,928-foot viaduct contained 870 precast segments, each weighing between 45 and 80 tons.

A bridge built in 1970 over the Loire River in Bloix, France, is shown on page 129. This is a two-spline segmental precast type bridge with polymer mortar joints; spans are nearly 300 feet. The bridge was erected in less than 6 months.

In summary, it is obvious that there is much activity concerning concrete polymer composites, both in the United States and abroad. Highway and bridge designers are now probing potential uses of these materials in experimental projects. The Brookhaven researchers have indicated that the time is appropriate to start designing and constructing polymer impregnated concrete structures and bridge members, and several State highway departments have indicated their interest in this area.

ACKNOWLEDGEMENT

The author wishes to express his appreciation to Mr. Meyer Steinberg of the Brookhaven National Laboratory for furnishing the illustrations denoting the material properties and laboratory tests of polymer impregnated concrete and for reviewing this paper.

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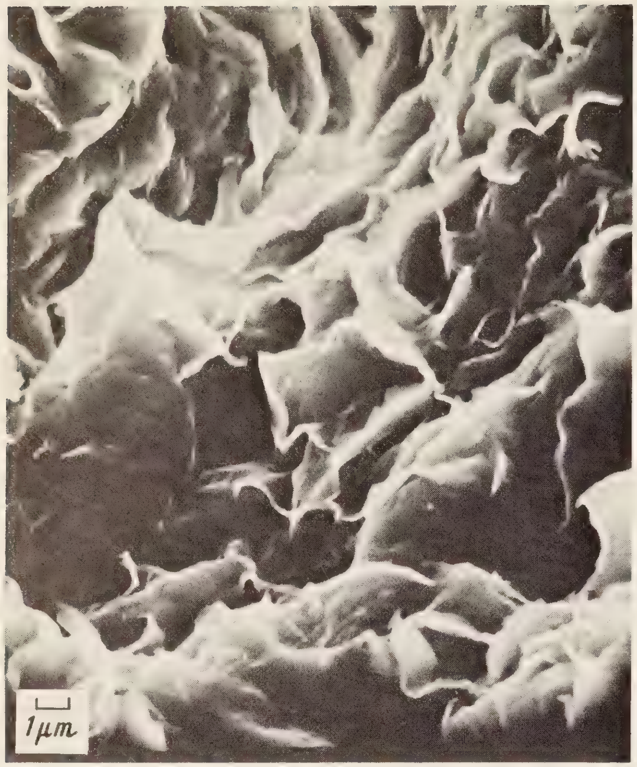

Scanning electron micrograph of untreated montmorillonite.

Strength Development and Reaction Products in Lime-Montmorillonite-Water Systems

Reported by W. C. ORMSBY,
Research Chemist, and
E. B. KINTER, Highway
Research Engineer,
Materials Division

OFFICE OF RESEARCH

Results of a study which, overall, concerned the physical and rheological properties and basic strength-promoting reactions in lime-clay-water systems are reported in this article. The part of the study related to the strength development and reaction products in calcitic lime-montmorillonite-water systems is represented here. Other results obtained in the study are reported in an article entitled "Effects of Dolomitic and Calcitic Limes on Strength Development in Mixtures with Two Clay Minerals," page 149.

Previously published reports dealt with the physical properties and reaction products in kaolinite-lime-water systems (1, 2),[1] and the rheological properties of lime-montmorillonite-water systems (3).

The effects of treating size-fractionated, calcium-saturated montmorillonite with hydrated lime were investigated by measuring unconfined compressive strength. Mineralogical changes effected by lime treatment were studied using chemical and X-ray diffraction analysis. Lime applications and curing procedures were designed to simulate actual field conditions.

Results indicate that the stabilization reactions occurring between lime and montmorillonite are rapid and substantial and form an amorphous or very poorly crystallized product identified as calcium silicate hydrate gel (CSH (gel)) but with only limited degradation of the clay. Since the amounts of the reaction product could not be measured, only a general correlation between reaction product development and strength could be obtained.

Introduction

INVESTIGATIONS of the effects of hydrated lime on the strength properties and reaction products in various clay-water systems have been numerous and varied since the pioneer work of Eades (*4*) and Eades and Grim (*5*). Pertinent findings from recent investigations are summarized in the papers of Mateos (*6*), Diamond and Kinter (*7*), Ingles (*8*), and Thompson (*9*). As noted by Moh (*10*) and Stocker (*11*) many investigations have been conducted under various exaggerated conditions, i.e., extremely high lime contents, unrealistically high curing temperatures, and very high water contents (lime-clay-water suspensions). Consequently, there is an understandable lack of agreement as to the identity of reaction products and the course of their development in lime-stabilized clays. Further, because such conditions differ widely from those normal to highway construction and service life, there is also considerable uncertaintly in applying the findings to interpret the results of practical soil-lime treatment.

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[1] Italic numbers in parentheses identify the references listed on page 148.

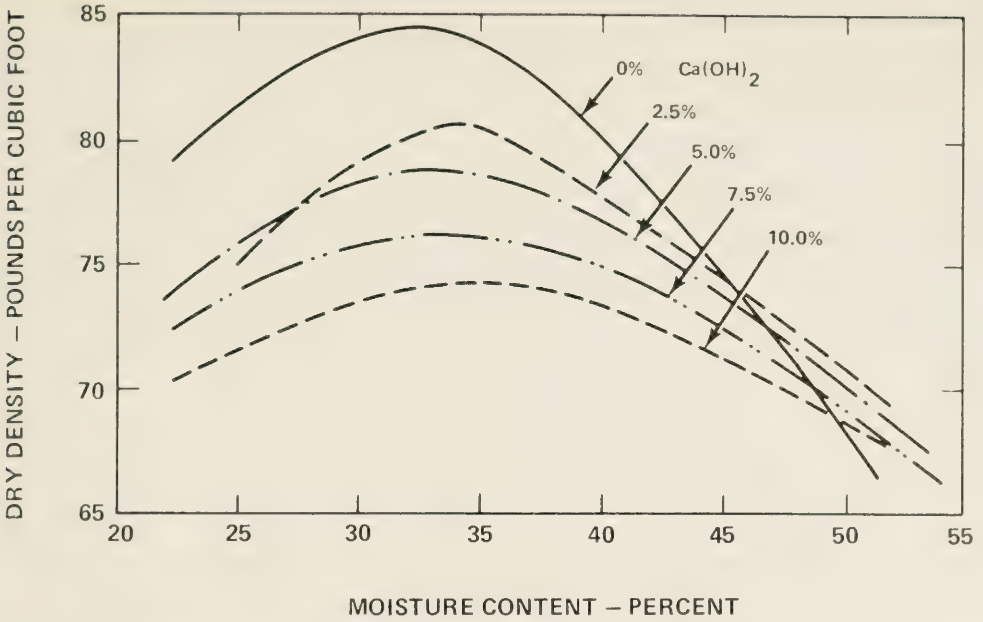

Figure 1.—Moisture-density curves for montmorillonite and montmorillonite-calcitic lime mixtures.

In this study, strength properties and reaction products were investigated in lime-montmorillonite-water systems in which reaction conditions were controlled to approximate actual highway environments. Practical lime contents were used and lime-clay-water mixtures were compacted to maximum density at optimum moisture content (densities approximated those obtainable on full-size specimens compacted by AASHO T 180). Specimens were made by compacting mixtures both with and without prior loose mellowing. Curing periods of 7 and 28 days at ambient temperature were utilized. Slightly elevated temperatures were used to simulate longer periods of ambient temperature curing.

Experimental Methods

Materials

The fraction of montmorillonite smaller than 2 micrometers (—2 μm) obtained from a commercially available Wyoming bentonite was used. Coarser impurities were removed by gravitational separation after the crude bentonite was dispersed in water. Using centrifugation techniques, the —2 μm clay was saturated with calcium by four treatments with 1N CaCl$_2$, and washed with water-acetone solution until free from chloride, as indicated by the silver nitrate test. The washed clay was dried at 80°C., pulverized to pass a No. 80 sieve, and stored in sealed glass jars until used for preparing mixtures and specimens.

A finely powdered, reagent grade Ca(OH)$_2$ (Blaine fineness of 4,330 sq. cm./g) was used for preparing lime-clay-water mixtures at five levels of lime content: 0, 2.5, 5.0, 7.5, and 10.0 percent, based on the 110°C. dry weight of the clay.

Moisture-density relations

Moisture-density relations of the pulverized clay and various lime-clay mixtures were determined by the use of miniaturized compaction equipment previously developed (1). In each moisture-density determination, the clay-water or lime-clay-water mixture was dynamically compacted in three layers; the compacted densities obtained were in close agreement with densities obtained on full-size specimens of similar material compacted using AASHO test method T 180. Plots of density vs. moisture content are presented in figure 1. As expected, the curves show an increase in optimum moisture content and a decrease in density as the percentage of lime is increased.

Mixture preparation

In preparing mixtures, the required weight of clay or lime-clay combination (pre-mixed dry on glazed paper) was placed in a small plastic bag, the calculated amount of distilled water was added, and the bag was quickly sealed and thoroughly kneaded by hand. Some of the resulting loose mixtures were used in determining moisture-density relations, as indicated earlier; others were used to prepare sets of replicated compacted specimens for strength testing, X-ray diffraction examination, scanning electron microscope examination, and determination of residual free lime and the extent of carbonation.

Specimen preparation and curing regimes

Strength specimens were fabricated from the mixtures using the same miniaturized compaction equipment that was employed for the moisture-density tests. The specimens were compacted either (1) immediately after mixing was completed (no mellowing period) or (2) after the mixture had mellowed in the sealed plastic bag for 48 hours. Compacted specimens were immediately sealed in small plastic vials to minimize moisture loss and carbonation during curing. The variables involved in the mellowing and curing regimes are shown in table 1.

Table 1.—Descriptions of curing regimes, each regime beginning immediately after mixture preparation

Treatment No.	Mellowing period of loose mixture		Curing period of compacted specimens before testing or analysis				
	None	48 hours	None	48 hrs. at 120°F.	72 hrs. at 140°F.	7 days at room temperature	28 days at room temperature
1		X	X				
2		X		X			
3		X			X		
4	X		X				
5	X					X	
6	X						X
7	X			X			
8	X				X		

Strength tests

After curing, strength specimens were compressively loaded to failure in a commercial table model testing device (Instron tester Model TM) having a load sensitivity of 10 grams, and a strain rate of 0.10 inches per minute and a chart speed of 0.2 inches per minute. Unconfined compressive strength was computed from the rupture load and the cross-sectional area of the specimen. Immediately after each specimen was tested, it was weighed, dried at 110°C., and reweighed to provide data for computing the actual moisture content and density.

Determination of free lime and carbonation

To determine carbon dioxide and free (unreacted) lime, separate replicate specimens were prepared, and the compacted specimens, following the curing treatment, were dried 24 hours at 110°C. Free lime was determined by ASTM Method C 114-67 (*12*). In this method the free lime in a sample is solubilized by treatment with a hot solution of glycerol and alcohol and is subsequently titrated with an alcoholic solution of ammonium acetate. Carbon dioxide was determined by a modified version of the method of Shapiro and Brannock (*13*). In this method a dried, powdered sample is weighed and placed in the bottom of a test tube having a previously calibrated side arm. Mineral oil is added and the volume of carbon dioxide produced by the addition of hydrochloric acid (1:1) is measured. From the calibration, the volume of carbon dioxide liberated and collected in the side arm is readily converted to percent carbon dioxide.

In order to assess the effects of the oven drying procedure on the disposition of lime in the cured specimens, separate sets of specimens were prepared and placed in a vacuum desiccator immediately after compaction and curing. Vacuum drying was effected at room temperature over magnesium perchlorate and ascarite (the desiccator being evacuated through a dry ice trap by means of a mechanical pump) for a period of 20 hours.

X-ray diffraction analysis

Specimens intended for X-ray diffraction analysis were cured and vacuum dried as previously described; they were retained under vacuum in the desiccator until required for use. The specimens were then pulverized to pass a No. 80 sieve, immediately treated with 10 percent glycerol in alcohol solution to form a thin paste, transferred by spatula to a glass microscope slide, and the X-ray diffraction pattern recorded. Glyceration was used in order to fix clay basal peak positions and enhance peak heights thus preventing possible overlap of the clay peaks and CSH (gel) peaks. Patterns made of the glycerated lime-clay mixtures were, except for the position and intensity of the clay basal peaks, identical to patterns made from dry pulverized lime-clay mixtures.

Mixtures representing all compositions and curing treatments were X-rayed, but patterns are given for only selected compositions.

Scanning electron microscopy

Specimens for scanning electron microscopy (SEM) examination and energy-dispersive X-ray analysis were cured and vacuum dried as described in the preceding paragraphs. The specimens were then fractured perpendicular to the altitude of the cylinder. The fractured specimen was then glued to a SEM specimen stub with silver paste and coated successively with carbon and gold-palladium in a vacuum evaporator. After this coating, the microstructure of the surface was studied in a Cambridge Stereoscan Model S–4 scanning electron microscope (fig. 2) using the secondary electron mode for imaging. The microscope was operated at an accelerating voltage of 20 kilovolts. In addition to secondary electron imaging, the energy-dispersive X-ray analysis accessory (left-hand portion of fig. 2) was used to determine elemental composition and elemental distribution in the specimens.

Results

Strength development

Data obtained from the unconfined compressive strength tests are given in table 2. These data were subjected to statistical analysis (tables 3 and 4), and the curves of strength vs. lime content, presented in figures 3 and 4, were obtained using regression equations. Points shown in the plots reflect the actual experimental values.

The five curves in figure 3 show strength-lime concentration relations for specimens which had not been mellowed. Differences in strength for specimens containing no lime and subjected to different intensities of cure can be attributed to thixotropic hardening. The minima in curves B, C, D, and E are due, at least in part, to the fact that the zero percent

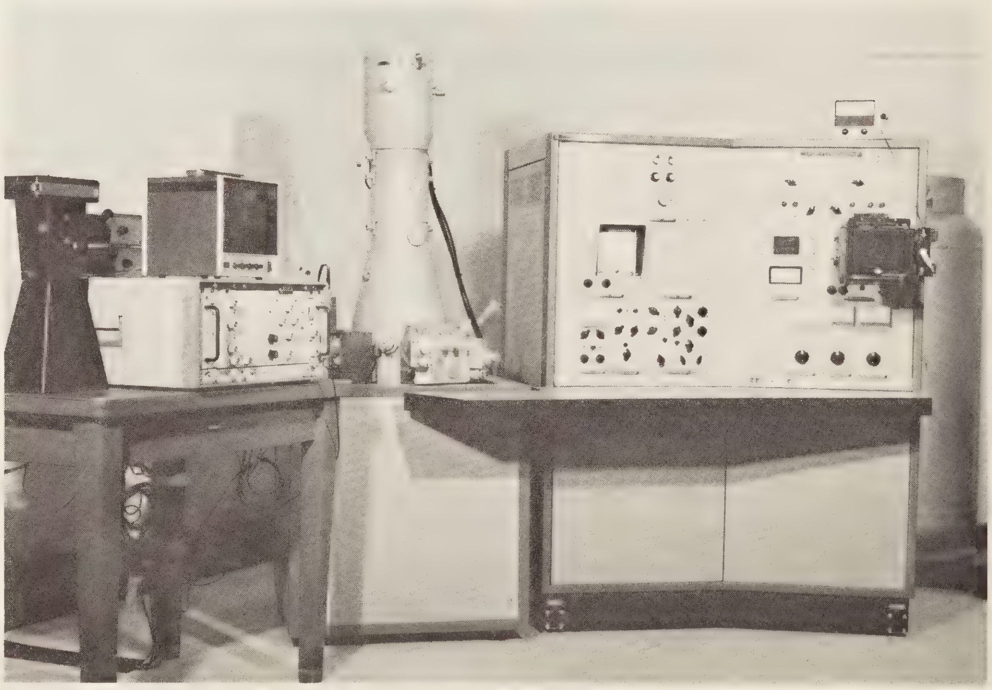

Figure 2.—Scanning electron microscope equipped with energy-dispersive X-ray analysis accessory.

Table 2.—Strengths of calcium-saturated Wyoming montmorillonite specimens treated with varying percentages of lime

Concentration of $Ca(OH)_2$ added to clay specimens (percent)	Unconfined compressive strength (p.s.i.)[1]							
	Curing treatment							
	Unmellowed					Mellowed		
	Uncured	7 days at room temperature	28 days at room temperature	48 hours at 120°F.	72 hours at 140°F.	Uncured	48 hours at 120°F.	72 hours at 140°F.
0	182	197	182	224	238	214	225	224
2.5	225	157	143	182	208	199	181	188
5.0	249	171	170	202	217	213	223	217
7.5	251	193	203	256	280	205	263	283
10.0	245	188	284	334	384	203	393	411

[1] Each strength value is the average of eight replicates.

lime compositions exhibit the thixotropic hardening effect whereas the beginning of particle to particle cementation from even a small percentage of lime effectively nullifies any thixotropic tendencies.

All levels of lime increased the strength over the lime-free composition for the unmellowed, uncured specimens (curve A). These strength increases (curve A), in the first few minutes after mixing, occurred before a significant quantity of cementitious reaction products could be formed and are therefore felt to be due to an increase in granularity of the system and a material response approaching elastic behavior. The development of this type of granularity was noted, described, and measured in earlier work by Ormsby and Diamond (*3*).

Figure 4 gives results for compositions which had been mellowed 48 hours prior

Table 3.—Results of regression analysis and analysis of variance for regression

Curing treatment	Degree	Reduction in sum of squares from fitting[1]	Standard error for fitting (p.s.i.)	Average standard error computed from replicates (p.s.i.)	Index of determination (r^2)	Best fitting equation
Mellowed, uncured	0	....	6.50	4.18	0	$Y=206.8$
	1	n.s.	6.90		0.15	
	2	n.s.	8.40		.16	
Mellowed, cured 48 hrs. at 120°F.	1	**	54.80	8.16	.66	$Y=223-23X+4X^2$
	2	*	15.24		.982	
	3	n.s.	21.50		.9825	
Mellowed, cured 72 hrs. at 140°F.	1	**	56.44	9.57	.70	$Y=223-24X+4.2X^2$
	2	**	4.53		.99+	
	3	n.s.	6.38		.99+	
Unmellowed, uncured	1	**	18.66	7.87	.69	$Y=183+20X-1.4X^2$
	2	**	2.85		.995	
	3	n.s.	2.03		.999	
Unmellowed, cured at room temperature for 7 days	1	*	19.08	4.96	.03	$Y=197-31X+7.4X^2-0.4X^3$
	2	**	18.14		.42	
	3	*	1.31		.998	
Unmellowed, cured 28 days at room temperature	1	**	38.74	6.26	.607	$Y=178-18X+2.8X^2$
	2	**	9.51		.984	
	3	n.s.	12.19		.987	
Unmellowed, cured 48 hrs. at 120°F.	1	**	42.86	5.99	.61	$Y=220-19.6X+3X^2$
	2	**	8.63		.989	
	3	*	2.15		.999	
Unmellowed, cured 72 hrs. at 140°F.	1	**	49.72	7.62	.64	$Y=239-22X+3.7X^2$
	2	**	2.41		.99+	
	3	n.s.	3.34		.99+	

[1] **—significant at the 1 percent level; *—significant at the 5 percent level; n.s.—not significant.

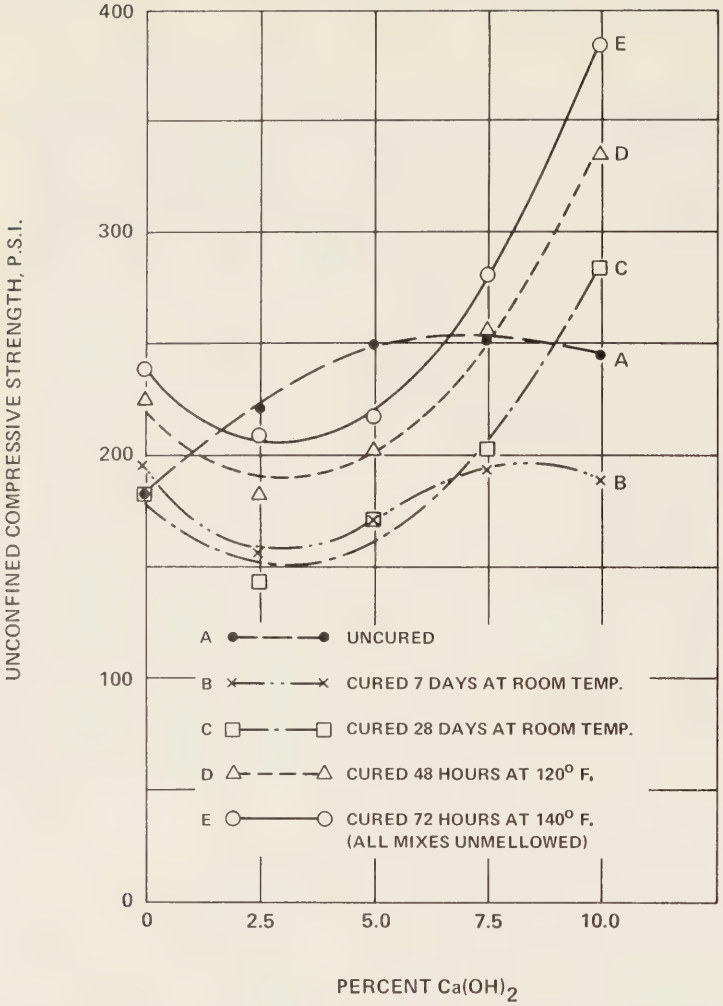

Figure 3.—Relations between unconfined compressive strength and percent lime for Ca-saturated Wyoming montmorillonite subjected to various curing regimes.

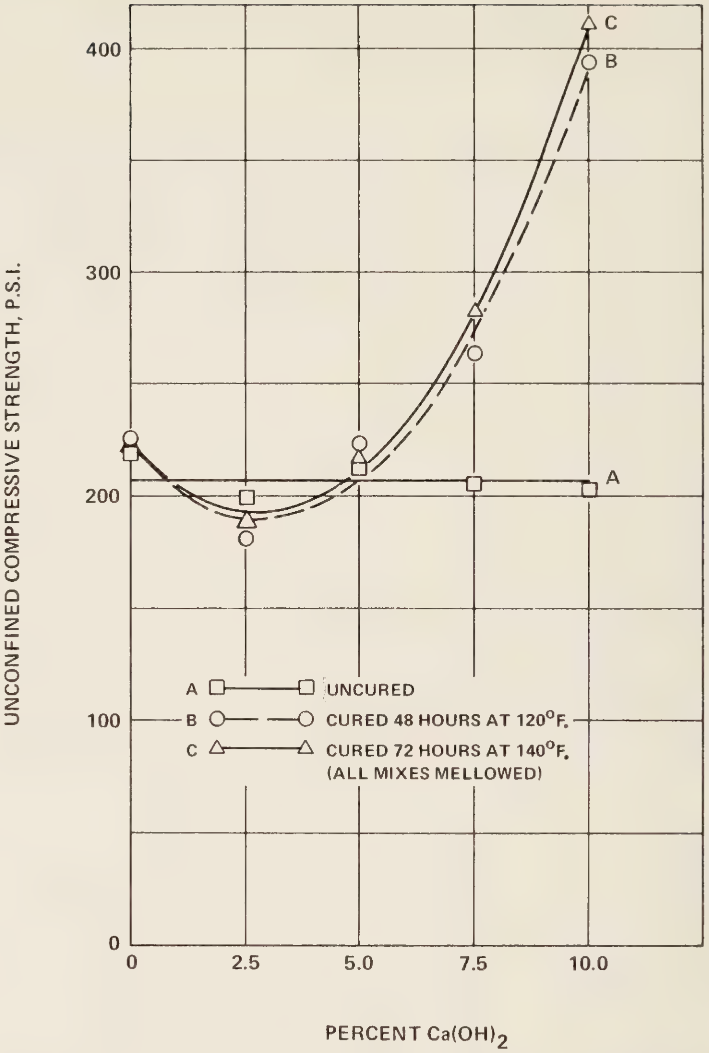

Figure 4.—Relations between unconfined compressive strength and percent lime for Ca-saturated Wyoming montmorillonite subjected to various curing regimes.

to fabrication and testing or curing. At all levels of lime mellowing noticeably lowered the strength of all uncured specimens (cf. fig. 3, curve A). The best fitting curve indicates that with mellowing, lime has no effect on strength. Mellowed, cured samples gave strength-lime concentration curves which were very similar to those obtained for corresponding unmellowed, cured samples.

In the process of evaluating the unconfined compressive strengths of the various compositions, complete load-time (equivalent to stress-strain) curves were recorded for the test specimens. These curves were found to be a good indicator of changes in behavior which lime additions and curing treatments were effecting in the various systems studied. In general it was found, in agreement with Thompson's work (*14*), that going from an untreated to a lime-treated clay there is an increase in the elastic character (brittleness) of the material. Tracings of load-time curves from some of the strength experiments are given in figures 5–11. These curves serve to illustrate several points:

Table 4.—Analysis of variance for regression—regression data obtained from least squares fitting of polynomials to strength results obtained from specimens which were mellowed and cured 72 hours at 140°F.

Source of estimate	Sum of squares	Degree of freedom	Mean square	F ratio[1]
Linear	22,087	1	22,087	1,699**
2d degree	9,425	1	9,425	725**
3d degree[2]	1	1	1	0.05 n.s.
Residual	40	2	20	
Total	31,553			

[1] **—significant at 1 percent level; n.s.—not significant.

[2] Variance for 3d degree pooled with residual.

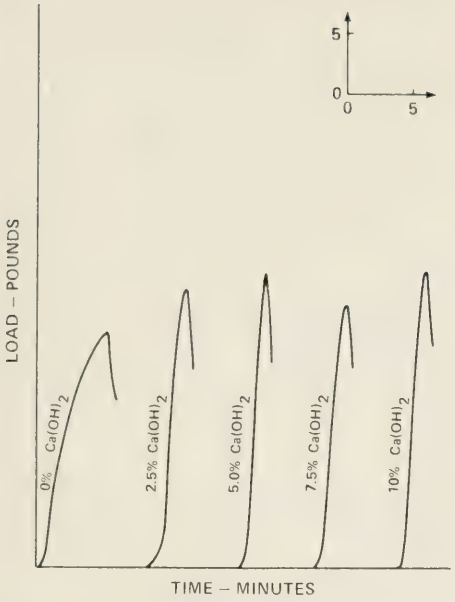

Figure 5.—Load-time curves for Ca-saturated Wyoming montmorillonite specimens treated with varying percentages of lime. Specimens unmellowed, uncured.

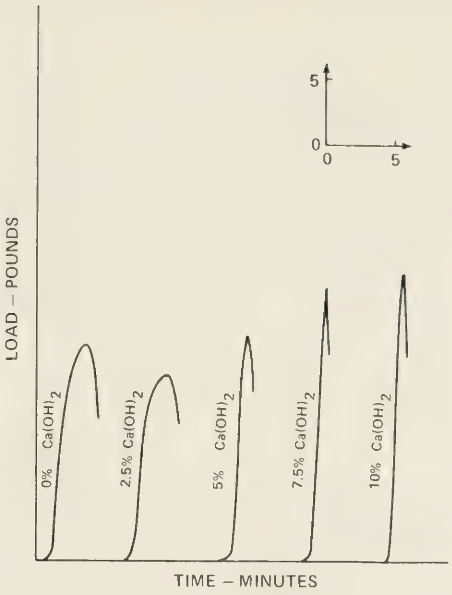

Figure 6.—Load-time curves for Ca-saturated Wyoming montmorillonite specimens treated with varying percentages of lime. Specimens unmellowed, cured 7 days at room temperature.

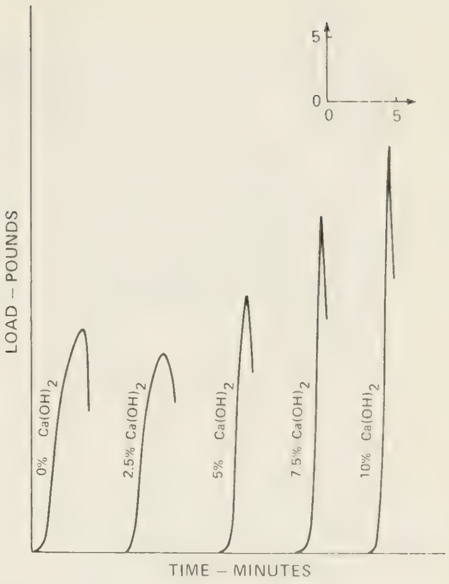

Figure 7.—Load-time curves for Ca-saturated Wyoming montmorillonite specimens treated with varying percentages of lime. Specimens unmellowed, cured 48 hours at 120°F.

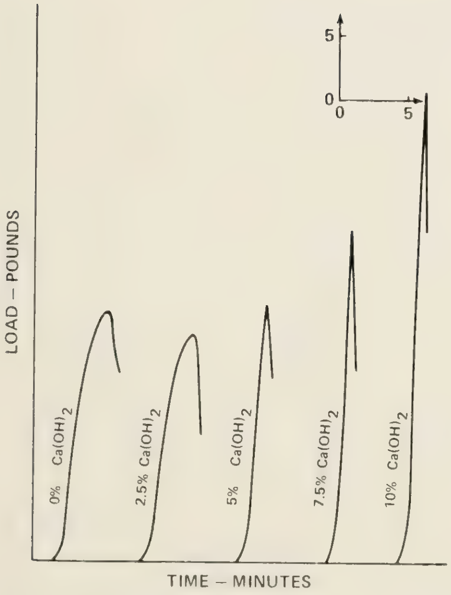

Figure 8.—Load-time curves for Ca-saturated Wyoming montmorillonite specimens treated with varying percentages of lime. Specimens unmellowed, cured 72 hours at 140°F.

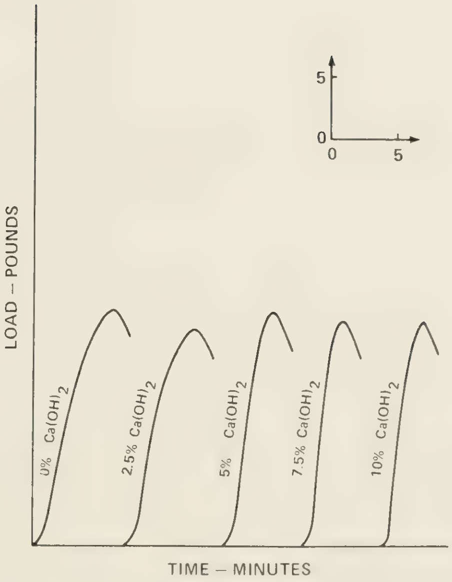

Figure 9.—Load-time curves for Ca-saturated Wyoming montmorillonite specimens treated with varying percentages of lime. Specimens mellowed 48 hours, uncured.

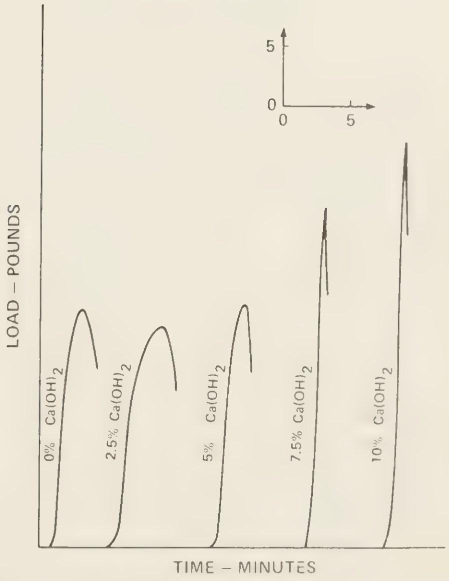

Figure 10.—Load-time curves for Ca-saturated Wyoming montmorillonite specimens treated with varying percentages of lime. Specimens mellowed 48 hours, cured 48 hours at 120°F.

Table 5.—Behavior of lime-montmorillonite-water mixtures fabricated at moisture contents for maximum density

Concentration of $Ca(OH)_2$ added to clay specimens (percent)	Curing treatment							
	Unmellowed					Mellowed		
	Uncured	7 days at room temperature	28 days at room temperature	48 hours at 120°F.	72 hours at 140°F.	Uncured	48 hours at 120°F.	72 hours at 140°F.
0	plastic	plastic	plastic	plastic	plastic	plastic	plastic	plastic
2.5	elastic[1]	do	do	do	do	do	do	do
5.0	do	elastic	elastic	elastic	elastic	do	do	do
7.5	do	elastic[1]	elastic[1]	elastic[1]	elastic[1]	do	elastic[1]	elastic[1]
10.0	do	do	do	do	do	do	do	do

[1] Strength improvement.

(1) Thixotropic hardening of lime-free compositions increases with increased intensity of cure (figs. 5–8 and 9–11);

(2) In general, as curing intensity is increased, increases in strength as well as a tendency toward more elastic behavior will occur (figs. 6–8 and 10 and 11);

(3) The elastic character which may be effected by modest lime treatment may, in cured specimens, be insufficient to offset the plastic and thixotropic nature of the clay (compare curves for zero and 2.5 percent lime in figs. 6–8 and 10 and 11);

(4) Montmorillonite-lime mixtures which are mixed and tested without mellowing or curing behave elastically (fig. 5); mixtures which are mellowed but not cured behave plastically (figure 9); and

(5) Cured montmorillonite-lime mixtures containing 5 or more percent lime behave elastically (figs. 6–8 and 10 and 11).

A summary of the behavior of the various systems tested is given in table 5. It is obvious that the deformation characteristics of the various systems can, in general, be qualitatively correlated with the strength characteristics as depicted in figures 3 and 4.

Disposition of lime in reacted mixtures

Carbon dioxide and free lime were determined for various lime-montmorillonite-water mixtures which duplicated many of the compositions and curing treatments used in the strength studies. Results of these tests are given in table 6 (columns 3 to 6). From column 4 of table 6, it is apparent that vacuum drying has minimized carbonation, keeping it at a very low level. With the exception of the 2.5 percent lime addition, the free lime contents of the vacuum-dried samples were consistently higher than those

Table 6.—Free lime and carbonate contents of oven-dried and vacuum-dried compacted clay-lime specimens

Curing treatment	$Ca(OH)_2$ added (percent)	$Ca(OH)_2$ carbonated (percent)		$Ca(OH)_2$ free (percent)		$Ca(OH)_2$ reacted (percent)[1]
		Oven	Vacuum[2]	Oven	Vacuum[3]	Vacuum
Unmellowed, uncured	2.5	0.30	0.14	1.04	0.89	1.47
	5.0	.70	.12	1.17	2.65	2.23
	7.5	1.20	.15	1.76	5.65	1.70
	10.0	1.30	.18	3.66	8.00	1.82
Mellowed, uncured	2.5	.40	.17	1.02	.46	1.87
	5.0	.80	.20	1.42	1.53	3.27
	7.5	1.10	.12	1.46	3.68	3.70
	10.0	1.30	.27	2.08	6.20	3.53
Mellowed, cured 48 hours at 120°F.	2.5	.30	.12	1.03	.59	1.79
	5.0	.50	.21	1.25	1.97	2.82
	7.5	.60	.21	1.58	2.76	4.54
	10.0	.90	.24	2.11	3.40	6.36
Mellowed, cured 72 hours at 140°F.	2.5	.40	.06	.96	.48	1.96
	5.0	.30	.07	1.20	1.83	3.10
	7.5	.50	.12	1.75	3.86	4.52
	10.0	.40	.17	2.28	3.10	6.73

[1] Calculated by difference.

[2] Standard error = 0.014.

[3] Standard deviation = 0.09.

of oven-dried samples. Apparently, oven drying produced some physical or chemical change (lime fixation) which precluded complete removal of uncombined calcium by glycerol-ethanol solvent used in the determination of free lime.

The amount of lime that had reacted with calcium montmorillonite was calculated as the difference between the amount added and the sum of the carbonated and the free lime. Column 7 of table 6 gives figures for the amount reacted as a function of curing treatment. Also, the amount of lime reacted as a function of the amount added and the curing regime is given in figure 12. These curves compare favorably with results obtained by Dumbleton (*15*) on heavy London clay.

For the unmellowed, uncured samples, significant quantities of lime were irreversibly removed (strongly adsorbed) at all lime levels.

Samples which were mellowed 48 hours but were uncured, compared to the unmellowed, uncured samples, adsorbed or reacted with additional lime. X-ray diffraction data (discussed later) indicate the formation of CSH (gel) in these systems. The ability of this reaction product to promote strength is nullified when the strength specimens are fabricated because cementitious bonds formed during loose curing are broken during the compaction process.

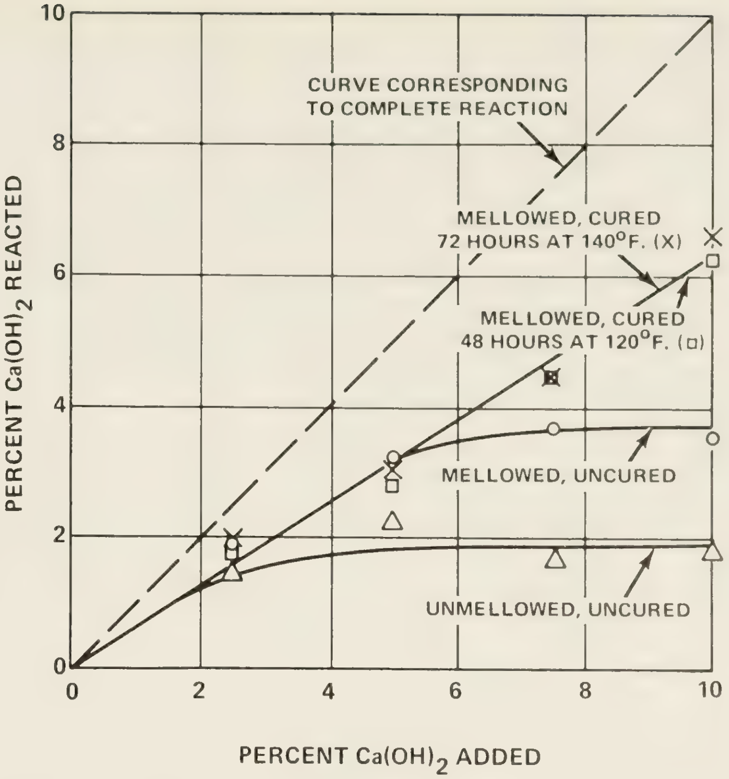

Figure 12.—Relation between proportion of lime added and proportion of lime reacted for Ca-saturated Wyoming montmorillonte treated with varying percentages of lime and cured under a variety of conditions.

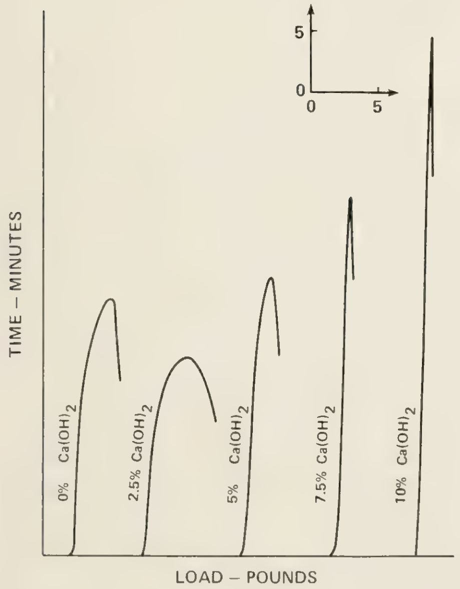

Figure 11.—Load-time curves for Ca-saturated Wyoming montmorillonite specimens treated with varying percentages of lime. Specimens mellowed 48 hours, cured 72 hours at 140°F.

Samples which were mellowed and oven cured showed increased lime consumption when compared to samples which were uncured. However, there was little difference in lime consumption between the two different oven-cured series.

X-ray diffraction studies

Except for the omission of the testing step and the substitution of vacuum drying for oven drying, specimens prepared for X-ray diffraction analysis received the same treatment as those used in the strength experiments. Selected X-ray diffraction patterns are shown in figures 13 and 14; the former figure gives patterns for samples of varying lime concentration cured at 140°F. for 72 hours while the latter gives patterns showing the effect of varying the curing conditions at a constant lime concentration.

For compositions cured at 140°F. for 72 hours, all except the sample containing 2.5 percent lime gave X-ray patterns indicating:

(1) Moderate degradation of clay peaks (note, e.g., decrease in intensity of clay basal peaks at two theta equal to 10°, 15°, 20°, 25°, and 30°),

(2) Development of a calcium silicate hydrate (CSH) reaction product (peak at two theta equal to 29.4°), and

(3) Some reaction between quartz contaminant (two theta equal to 26.6°) and lime.

None of the patterns showed peaks for calcium carbonate or free lime. The carbonate and free lime contents for these samples were quite low (see table 6) so that it is not too surprising that no peaks for these compounds were observed. Further, it is probable that the free lime in these samples was adsorbed or amorphous. No calcium aluminate hydrate (CAH) or calcium aluminate silicate hydrate (CASH) compounds were indicated.

For compositions containing 10 percent lime and cured under different conditions and excepting unmellowed, uncured samples, the X-ray patterns indicated:

(1) Moderate degradation of the clay peaks,

(2) Some reaction between lime and quartz,

(3) Development of a CSH product,

(4) No Ca(OH)$_2$ (free lime indicated by chemical analysis does not show up on X-ray pattern because it is too small in amount or is X-ray amorphous), and

(5) No CAH or CASH compounds.

Figure 15 gives smoothed diffractometer traces of a restricted two theta region for selected specimens. These traces show the growth of CSH X-ray activity at 29.4° two theta relative to the 006 montmorillonite peak (30° two theta) when the percent lime and/or the curing intensity is increased. These curves correlate qualitatively with the chemical analyses given in table 6 (see "Ca(OH)$_2$ reacted" column).

Diffraction patterns for samples representing other compositions and curing treatments (not included) indicated that CSH reaction product was being formed in all lime-containing samples except those which were unmellowed and uncured.

Scanning electron microscopy

Numerous specimens were examined in the scanning electron microscope; representative micrographs and calcium element X-ray distribution maps are shown in figure 16. This figure gives results for lime-free specimens and for specimens containing 10 percent lime which had been cured 48 hours at 120°F. These pictures typify results obtained in the SEM examinations. No morphologically discrete reaction products were found in these studies. The calcium "activity" indicated in figure 16(b) is from the calcium present as a result of calcium saturation. Figure 16(d), for a specimen containing 10 percent lime, shows that calcium (whether it is in the form of free lime or reacted lime) is rather uniformly distributed over the surfaces of the clay particles.

Discussion

Results of this investigation show that under normal or slightly accelerated cur-

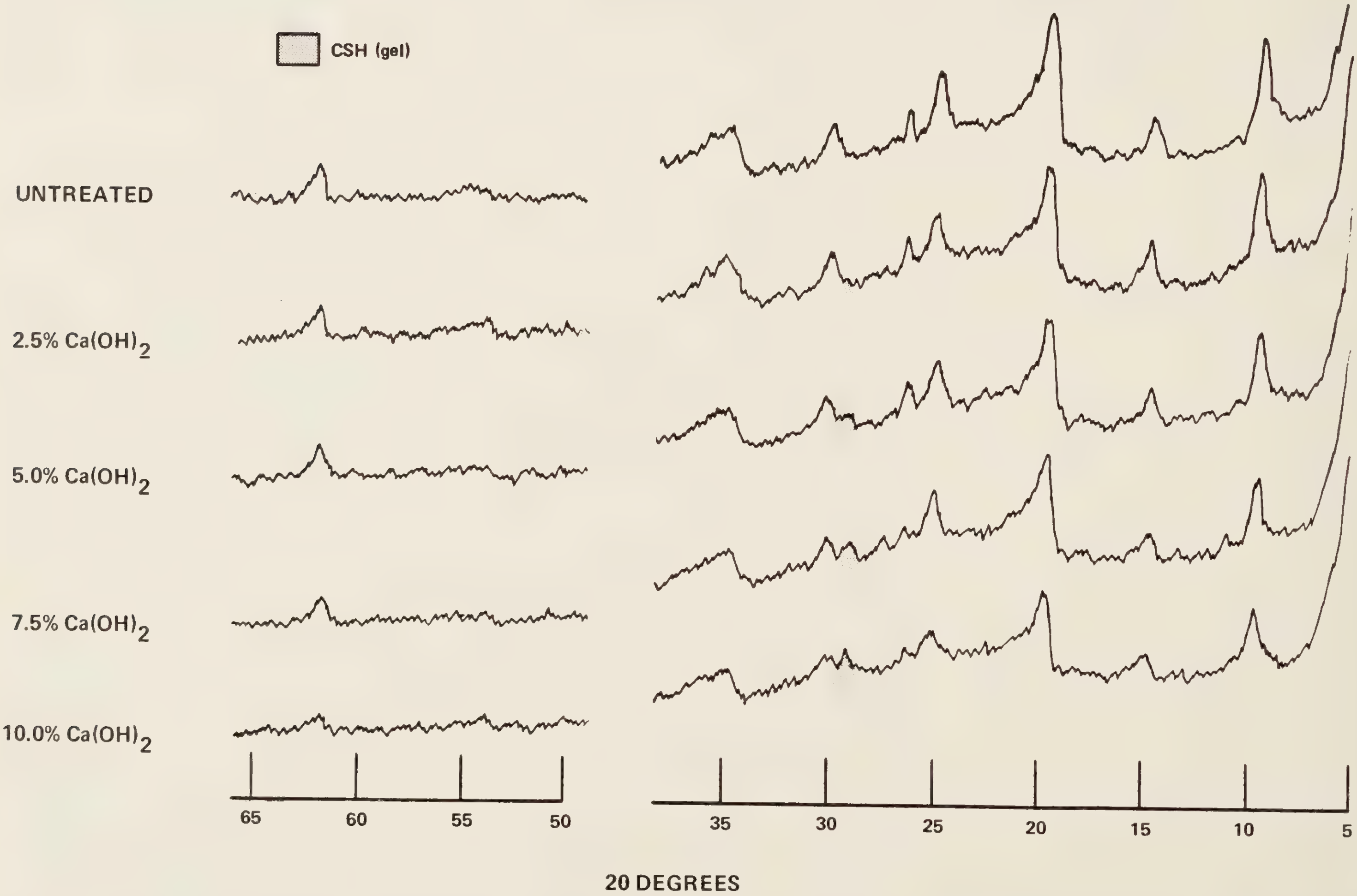

Figure 13.—X-ray diffractograms of Ca-saturated Wyoming montmorillonite specimens treated with varying percentages of lime, mellowed, and cured for 72 hours at 140°F.

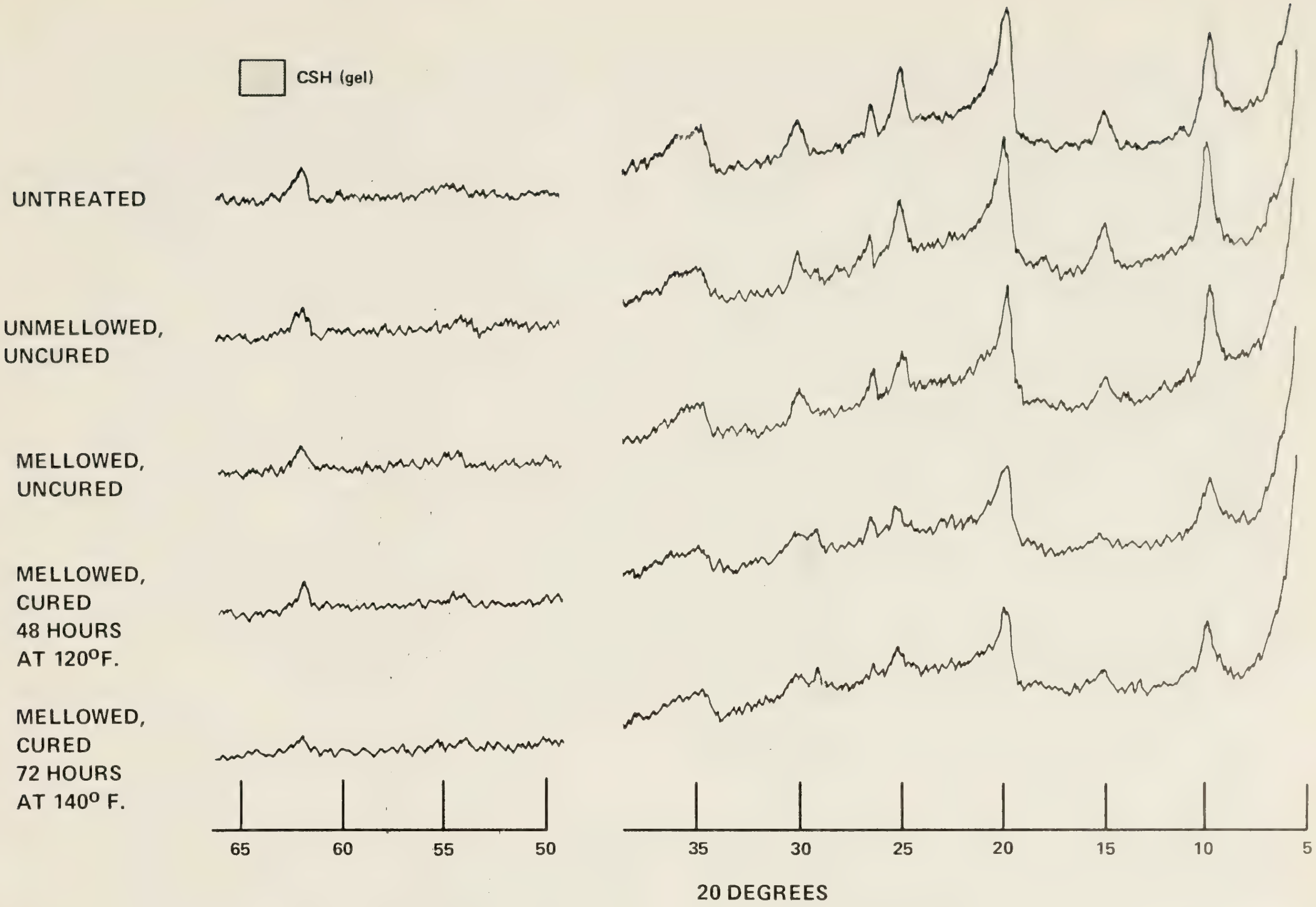

Figure 14.—X-ray diffractograms of Ca-saturated Wyoming montmorillonite specimens treated with 10 percent lime and cured under a variety of conditions.

ing conditions, there is a rapid and substantial interaction between lime and montmorillonite. X-ray diffraction and chemical analysis evidence revealed that while the rapid lime consumption resulted in the formation of a calcium silicate reaction product, considerable free (unreacted) lime that is X-ray undetectable (amorphous) remained in the system at the termination of the curing period. This latter material is believed to represent lime which has gone into solution and has either been adsorbed on the clay surfaces or has reprecipitated as X-ray amorphous lime. The reaction may be represented as follows:

$$\text{(C)AS}_4\text{H} + \text{CH} + \text{H} + \bar{\text{C}}\ \text{(atm.)} \longrightarrow \text{CSH(gel)} + \text{CH(adsorbed or amorphous)} + \text{C}\bar{\text{C}} + \text{unreacted (C)AS}_4\text{H}$$

$\text{(C)AS}_4\text{H}$ = calcium-saturated montmorillonite

C = CaO
A = Al_2O_3
CH = $Ca(OH)_2$
H = H_2O
$\bar{\text{C}}$ = CO_2
S = SiO_2
$\text{C}\bar{\text{C}}$ = $CaCO_3$
(C) = exchangeable Ca

Since other products, e.g., calcium aluminates or quaternary (CASH) compounds, were not detected by X-ray diffraction, the primary source of strength must lie with the calcium silicate product. The amount of CSH(gel) formed is small but substantial enough to promote strength development in cured specimens containing between 5 and 10 percent lime.

The exact mechanism by which the reaction product promotes strength is still open to speculation. Ingles (*16*) and Willoughby et al. (*17*) have suggested the development of "fabric coats" on clay or soil particles.

The evidence obtained in this investigation supports the hypothesis that in many soil-stabilizer systems where lime is found to improve strength characteristics, coatings may be closely involved in the strength development. In the scanning electron microscope work, examination of fracture surfaces of cured, compacted specimens of lime-montmorillonite-water mixtures indicates that the development of morphologically discrete reaction product particles of 200 Å or larger is rare. On the other hand, it appears from energy-dispersive (elemental) analysis of these fracture surfaces that the lime is rather uniformly disseminated throughout; no lime crystals, 200 Å or larger in size, were detected. It appears that both the free lime and clay-lime reaction products are more or less uniformly coating the particles or fabric units without the formation of new surface characteristics having distinct morphological identity.

Stocker's diffuse cementation theory *(11)* also seems to account for many stabilization effects; however, the reactions involved according to this theory, viz., topotactic edge reactions, do not, as previously indicated, seem to be generally supported by experimental data. Additionally, it is likely that if topotactic edge reactions were basic to the stabilization process, localized areas of morphologically unique, well developed reaction products would be readily seen during the examination of fracture surfaces of reacted specimens in the scanning electron microscope.

Summary

Unconfined compressive strength determinations and X-ray diffraction and chemical analyses were made of compacted specimens prepared from lime-montmorillonite-water mixtures in which lime concentration and curing treatments were varied. An attempt was made to correlate strength development with chemical and mineralogical changes. Results of the investigation are summarized as follows:

- In cured montmorillonite-lime mixtures prepared at optimum moisture content, reaction products responsible for strength improvements are CSH (gel) materials. Calcium aluminates or quaternary compounds were not detected.
- Reactions producing the CSH (gel) materials progress rapidly from the time of mixing, significant quantities of reaction product being found even in loose mixtures cured for a short period of time.
- The development of sufficient cementitious reaction products to significantly improve the strength of compacted specimens results in only a slight degradation of the montmorillonite crystal structure.
- Although some light has been shed on montmorillonite-lime reactions, and the relation between reaction products and strength development, the exact mechanisms are still only partly resolved. Evidence from chemical, scanning electron microscope and X-ray analyses suggests that added lime dissolves in the water phase, reacts with silica (and possibly alumina) of the clay forming poorly crystallized reaction products which are distributed over the surfaces of the clay particles in a presumably uniform layer that is less than 200 Å in thickness.

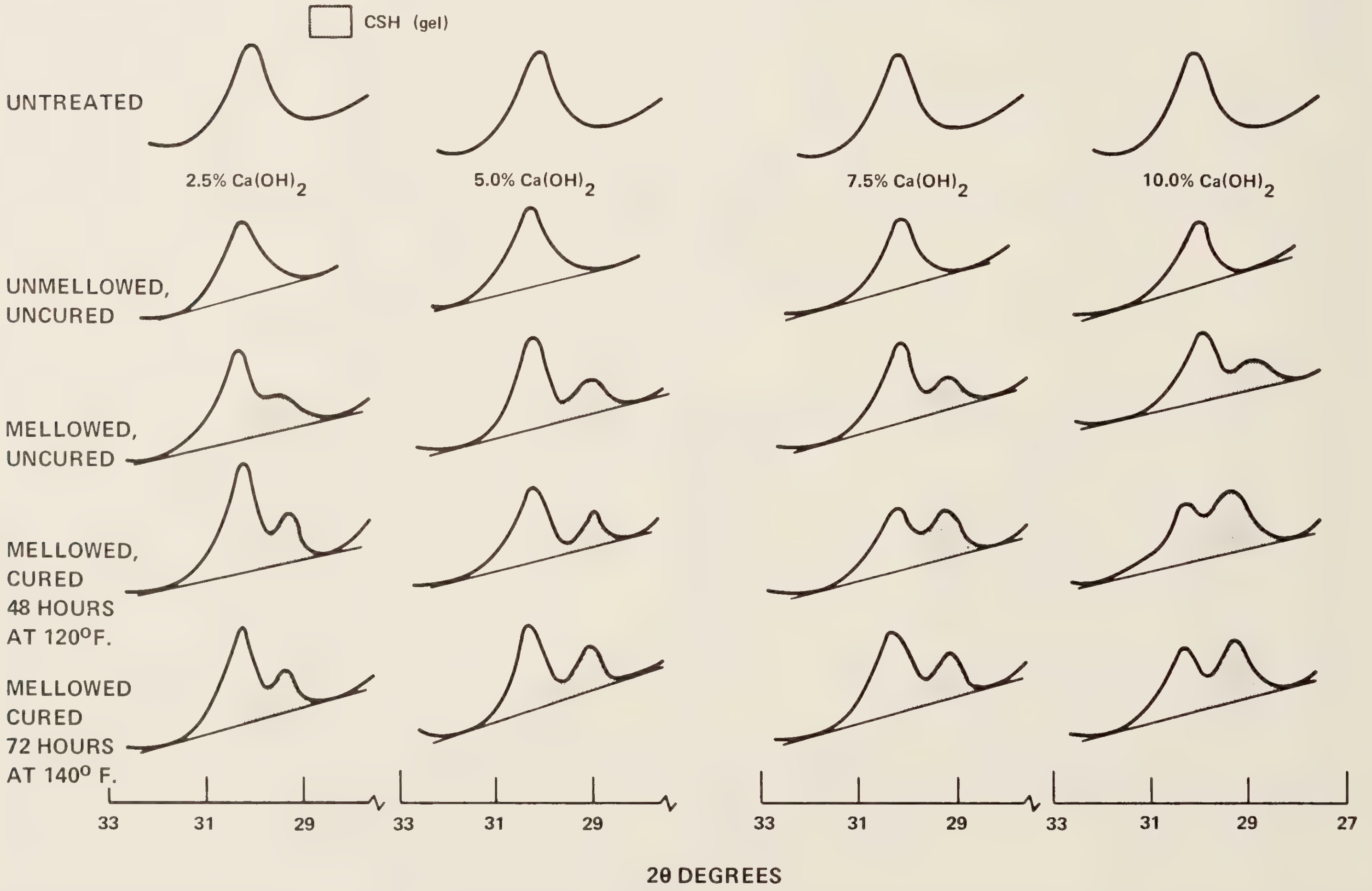

Figure 15.—High sensitivity X-ray diffractograms of restricted 2θ region of Ca-saturated Wyoming montmorillonite specimens; percent lime and curing conditions variable.

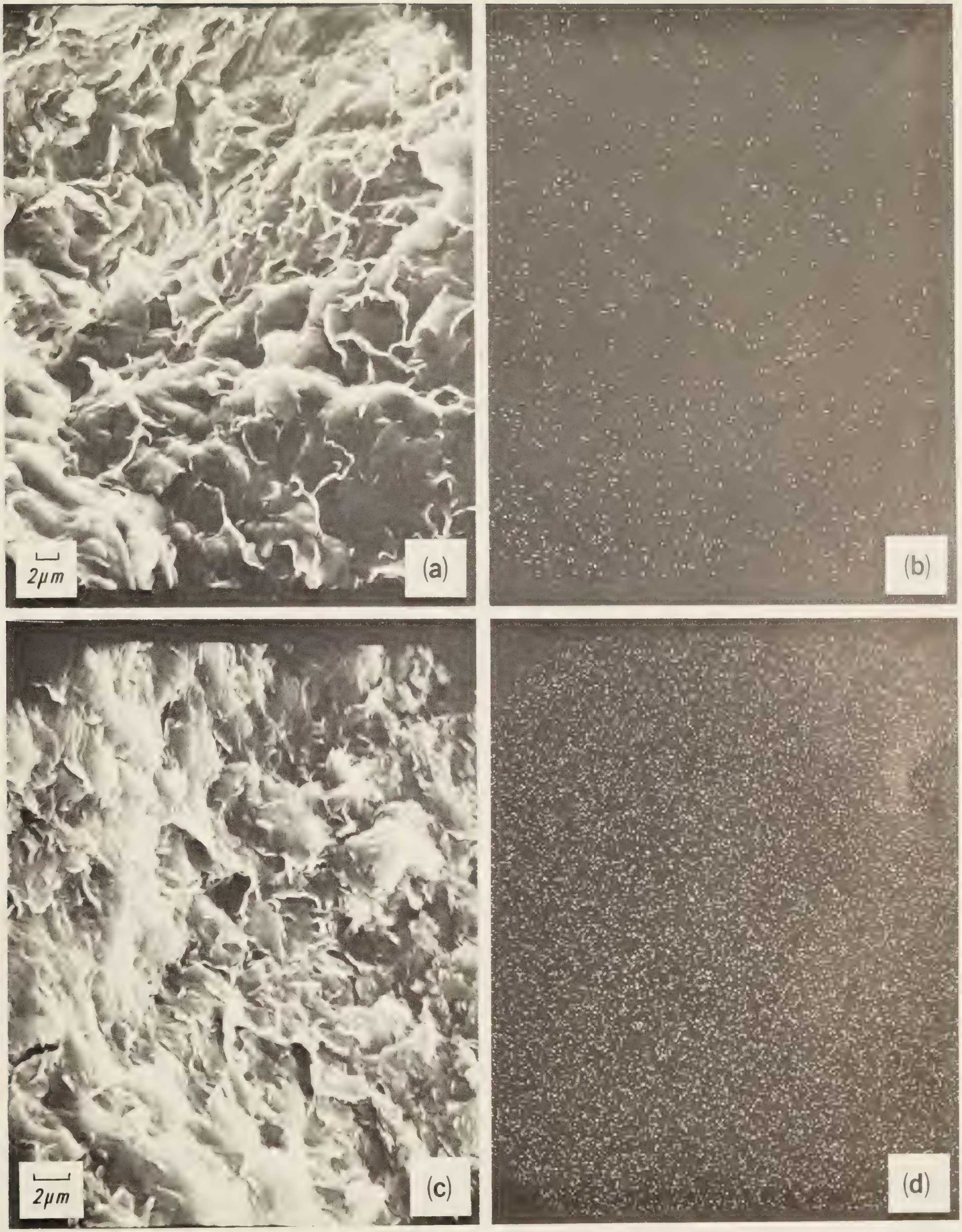

Figure 16.—Scanning electron micrographs and calcium element maps of untreated—(a) and (b)—and lime-treated—(c) and (d) —montmorillonite cured 48 hours at 120°F.

ACKNOWLEDGEMENTS

The authors express their thanks to E. F. Bailey, who assisted in all phases of the experimental work, and to Donald G. Fohs for valuable discussions.

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Errata

The following items appearing on page 65 of Public Roads, vol. 37, No. 2, September 1972, were published in error:

Hydraulic Analysis of Circular Culverts—Program HY–1 (PB 168715)

Hydraulic Analysis of Pipe-Arch Culverts—Program HY–2 (PB 168716)

Hydraulic Analysis of Box Culverts—Program HW–3 (PB 168717)

Hydraulics of Bridge Waterways—Program HY–4 (PB 168718)

Flood Record Compilation and Frequency Plot—Program HY–5 (PB 176539)

Effects of Dolomitic and Calcitic Limes on Strength Development in Mixtures with Two Clay Minerals

Reported by W. C. ORMSBY,
Research Chemist, and
E. B. KINTER, Highway Research
Engineer, Materials Division

OFFICE OF RESEARCH

Conflicting reports about the relative effectiveness of two principal lime types—dolomitic monohydrated and calcitic hydrated—in soil stabilization have resulted in a need to re-evaluate the response to lime stabilization of the reactive components in soils.

Samples of two clay minerals occurring widely in soils—kaolinite and montmorillonite—were size-fractionated, calcium-saturated, and lime-treated under a variety of experimental conditions. The response to the lime treatment was gaged by changes in the compressive strength of compacted specimens. Mineralogical changes in lime-clay mixtures were followed by X-ray diffraction analysis.

Results showed that the calcitic lime was much more effective than the dolomitic lime for improving the strength characteristics of kaolinite. With montmorillonite, however, the dolomitic lime was slightly superior when specimens were cured at room temperature, but the two limes produced essentially identical results at slightly elevated temperatures.

X-ray diffraction results indicated that different types of chemical reactions occurred with the two clay minerals. The bonding phase in mixtures containing montmorillonite and either of the two limes was a calcium silicate hydrate, CSH (gel); the bonding phase in the kaolinite-lime-water mixtures was an X-ray amorphous phase of indeterminate composition.

Introduction

IN 1961, Herrin and Mitchell (*1*)[1] noted the need for research on "the influence or effectiveness of various types of limes for stabilizing soils . . ." Ingles (*2*), 1962, stated, "Typical fields where further work promises important advances in knowledge of stabilizer behavior are, for lime stabilization, the study of dolomitic lime reactions in soil. . . ." Several prior as well as subsequent studies (*3–9*), involving a number of types of lime, have indicated that, under ambient conditions, in many soil-lime mixtures dolomitic monohydrated lime is more effective than calcitic hydrated lime in producing strength.

In spite of this seeming abundance of information, doubt has persisted as to the general effectiveness of dolomitic monohydrated lime in promoting strength in clays and clayey soils. For example, with montmorillonitic soils, Andrews and O'Flaherty (*10*) found that the dolomitic monohydrate was less effective than the calcitic hydrate, whereas Mateos (*11*) reported it to be generally more effective. With kaolinitic clay soils, Andrews and O'Flaherty found dolomitic monohydrated lime to be the more effective, but Mateos reported that the two limes were approximately equal in effectiveness.

In view of these uncertainties, this study was undertaken to obtain a clearer picture of the response of two important clays to treatment with these two lime types. The study concerned chiefly the unconfined compressive strength properties of two carefully purified clay minerals, kaolinite and montmorillonite, treated with practical percentages of the two limes and cured under a variety of conditions, including moist-curing at room temperature. By working with fractionated, calcium-saturated clays, the complicating effects of ion exchange and excessive mineral impurity content were eliminated. In addition to the measurement of unconfined compressive strength, the progress of the reactions in the clay-lime-water mixtures was followed by use of X-ray diffraction.

Experimental Methods

Materials

A Florida kaolin and a Wyoming montmorillonite (bentonite), were chosen for study. These commercial clays were size-fractionated and then calcium saturated by repeated treatments with excess 1N $CaCl_2$ solution and subsequent

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[1] Italic numbers in parentheses identify the references listed on page 160.

Table 1.—Properties of laboratory-treated clays

Property	Florida kaolin	Wyoming montmorillonite
Particle size (micrometers)	2–0.25	Less than 2
Exchange cation	Ca++	Ca++
Cation exchange capacity (m.e./100 g)	ca.10 (*12*)	ca.90 (*13*)
Mineral composition (X-ray diffraction)	kaolinite	montmorillonite (with small amount of quartz)
Hygroscopic moisture (percent)	0	ca.3

washing with alcohol/acetone solution until chloride-free.[2] After oven drying—kaolin at 110°C., montmorillonite at 80°C.—the clays were passed through a No. 80 sieve. Some properties of the clays are given in table 1.

Reagent grade $Ca(OH)_2$ and a 1:1 mixture (mole basis) of reagent grade MgO and $Ca(OH)_2$ were used to represent calcitic hydrated and dolomitic monohydrated limes, respectively. The $Ca(OH)_2$ and MgO had finenesses (Blaine) of 4,330 cm²/gm and 9,340 cm²/gm, respectively.

Moisture-density determinations

Moisture-density determinations were made on mixtures made from each clay treated with 0, 2.5, 5.0, 7.5, and 10.0 percent of $Ca(OH)_2$, using miniaturized equipment previously developed (*14*). Only calcitic lime was used in these determinations since previous study (*10*) had shown that lime type is not a major factor in causing variations in optimum moisture content or maximum dry density. Results of the determinations are given in figures 1 and 2.

Strength measurements

For strength and X-ray diffraction studies, clay-lime-water mixtures were prepared for each clay at optimum moisture content, with five levels of lime: 0, 2.5, 5.0, 7.5, and 10 percent, based on the 110°C., dry weight of clay. Miniature (0.334 in. diameter by 0.689 in. length) compacted cylindrical specimens were prepared from the mixtures to represent the following curing regimes:

(1) Specimens compacted immediately after mixture preparation, moist-cured 28 days at room temperature.

(2) Specimens compacted immediately after mixture preparation, moist-cured 2 days at 120°F.—equivalent to about 45 days at 70°F. (*15*).

(3) Loose mixture mellowed 48 hours at room temperature, specimens prepared and then moist-cured 2 days at 120°F.

Mixing and specimen fabrication procedures were identical to those previously described (*14*). Mixtures of clay, lime, and water were prepared in small plastic bags and when ready for compaction were removed and compacted dynamically using miniaturized equipment. Three layers were used and each layer was tamped eight times. This procedure produced compacted densities which correlated well with densities obtained on full-size specimens of similar material compacted using AASHO T 180 (*16*). For each lime

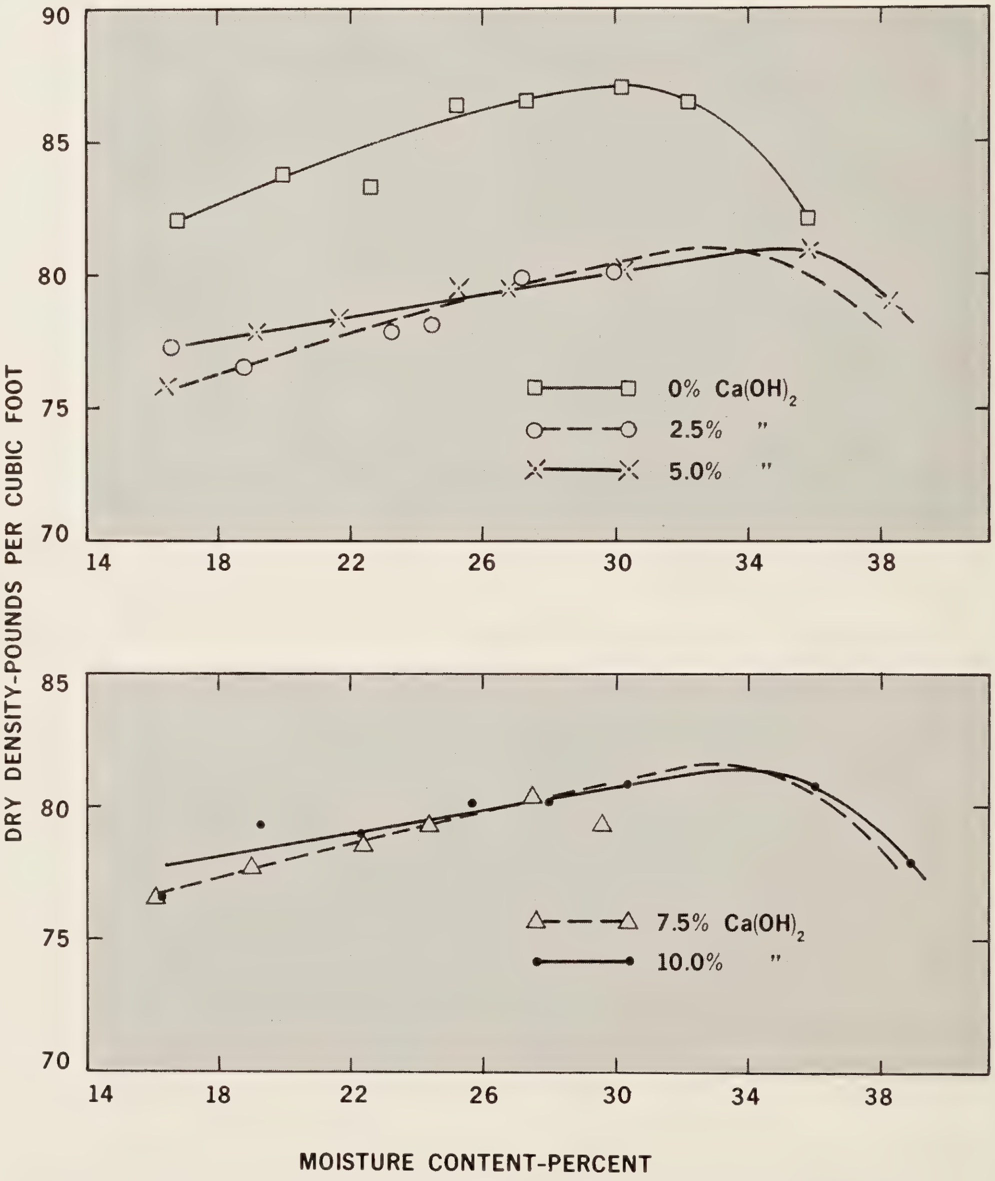

Figure 1.—Moisture-density curves for kaolinite and kaolinite-calcitic lime mixtures.

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[2] Because of the relative solubilities of $Ca(OH)_2$ (ca.0.02 g/100 ml) and MgO (0.00062 g/100 ml), the solution phases in all of the subsequently prepared clay-lime-water mixtures were saturated with respect to calcium. Therefore, there was no tendency for the exchangeably-held calcium ion to contribute to any reactions produced by the added lime.

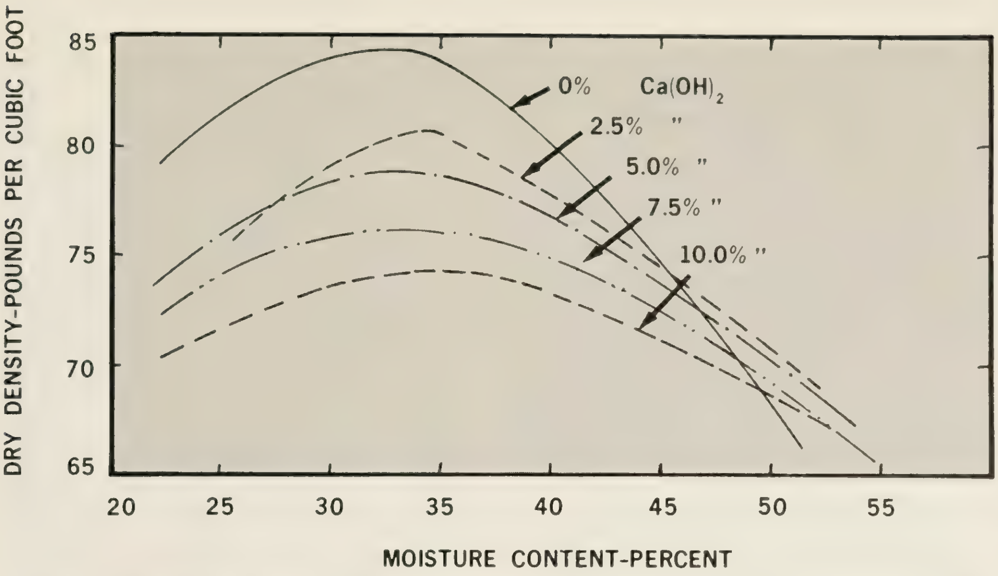

Figure 2.—Moisture-density curves for montmorillonite and montmorillonite-calcitic lime mixtures.

type, a set of 100 specimens was prepared: 10 replicates for each clay (2) at each lime percentage (5). Compacted specimens were tightly sealed in small plastic vials and cured by placing in a high-humidity chamber at room temperature or in an oven at 120°F.

After curing, eight of the replicates for a given treatment-curing situation were removed from the vials and quickly tested for unconfined compressive strength, using the apparatus and procedures previously described (*14*). Specimens were weighed immediately after testing, oven-dried at 110°C. and reweighed, to provide data for computations of dry densities and moisture contents. The latter data were used only as a check on attainment of prescribed density and moisture content, and are not included in this article.

X-ray diffraction analyses

Of the set of 10, the remaining two replicates were removed from their vials, immediately placed in a desiccator, and dried and retained under vacuum at room temperature until needed for X-ray and scanning electron microscope study. One of the specimens was then crushed to pass a No. 80 sieve and treated as follows: with mixtures containing montmorillonite the crushed material was solvated with glycerol and X-rayed on a glass microscope slide; with kaolinite-bearing mixtures, the crushed material was used to prepare dry powder X-ray specimens according to the method of McCreery (*17*).

Mixtures representing all of the compositions and curing treatments for both clays were X-rayed, but patterns are given for only the 10 percent compositions, as shown in the following paragraph.

Scanning electron microscope examination

Specimens for scanning electron microscope examination were prepared and examined as described on page 138 of this issue of *Public Roads.*

Results and Discussion

Data obtained from the strength tests are given in table 2. To aid in evaluating

Table 2.—Strength of calcium-saturated clays treated with varying percentages of calcitic and dolomitic monohydrated limes

Concentration of lime in soil-lime mixtures (percent)	Unconfined compressive strength (p.s.i.)[1]					
	Calcitic lime			Dolomitic lime		
	Moist-cured 28 days at room temperature [2]	Moist-cured 2 days at 120°F.[2]	Moist-cured 2 days at 120°F.[3]	Moist-cured 28 days at room temperature [2]	Moist-cured 2 days at 120°F.[2]	Moist-cured 2 days at 120°F.[3]
			Kaolinite			
0	252	235	217	252	233	233
2.5	354	396	354	297	306	308
5.0	418	636	533	405	457	465
7.5	529	946	838	426	555	577
10.0	615	1002	914	442	597	619
			Montmorillonite			
0	182	224	225	182	224	223
2.5	143	182	181	190	169	183
5.0	170	202	223	206	230	253
7.5	203	256	263	242	271	293
10.0	284	334	393	327	306	355

[1] Each strength value is the average of eight replicates.

[2] Specimens compacted immediately after mixture preparation—then cured.

[3] Loose mixture mellowed 48 hours at room temperature—then compacted and cured.

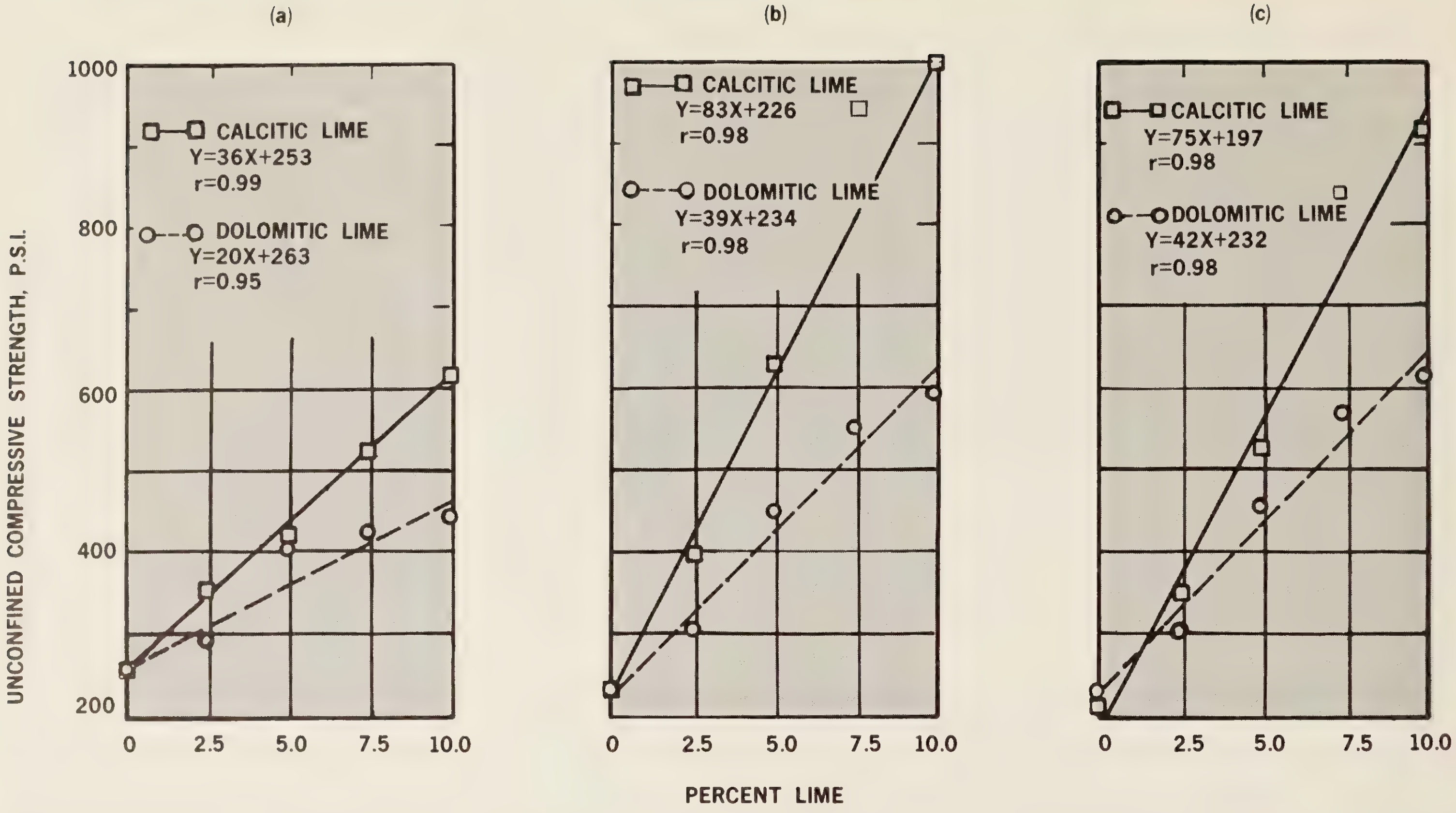

Figure 3.—Strength vs. lime concentration for kaolinite-lime mixtures: (a) Specimens compacted immediately after mixture preparation, moist-cured 28 days at room temperature. (b) Specimens compacted immediately after mixture preparation, moist-cured 2 days at 120°F. (equivalent to about 45 days at 70°F.) (c) Loose mixture mellowed 48 hours at room temperature, specimens prepared and then moist-cured 2 days at 120°F.

the effects of variations in lime type, lime concentration, and curing treatment, least-squares polynomials were fitted to the strength-lime concentration data. The results of this curve fitting are presented in figures 3 and 4. The strength response from the two limes is further illustrated in figure 5, which gives the difference in strength response (calcitic minus dolomitic) as a function of lime percentage.

X-ray diffractograms for kaolinite-10 percent calcitic and kaolinite-10 percent dolomitic lime compositions are presented in figures 6 and 7, respectively. Corresponding diffractograms for montmorillonite compositions are presented in figures 8 and 9, respectively.

Strength effects

The curves in figure 3 indicate that, with kaolinite, the calcitic lime is considerably more effective in producing strength than the dolomitic, both with room-temperature curing (a) and elevated-temperature curing (b) and (c). They also show that with kaolinite-calcitic lime mixtures loose mellowing prior to compaction slightly reduces strength development; with kaolinite-dolomitic lime mixtures mellowing before compaction had no measurable effect.

Corresponding curves for montmorillonite-lime mixtures, figure 4, indicate that the type of lime has little effect on strength development; with room-temperature curing (a) the dolomitic lime is slightly superior; with the elevated-temperature curing, however, there is no measurable difference between the two types. The curves also show that, with montmorillonite, a minimum addition of about 5 percent of lime is required before there is a gain in strength, and the level attained is lower than that attained with kaolinite. The latter two observations are in accord with the findings of other studies with montmorillonite-lime systems (*18*).

Other noteworthy features of the curves in figure 4 are as follows:

- There is a strength minimum at about 2.5 percent lime, an effect also noted by Eades and Grim (*19*), in work with a naturally occurring calcium-saturated montmorillonite.
- At zero-percent lime, specimen strength is greater for the higher temperature curing—figure 4(b) and (c)—than for room temperature curing (a). The latter effect may be the result of greater thixotropic hardening in specimens subjected to what is in effect a longer curing period.

The differences in strength response between the two types of lime are more clearly demonstrated in figure 5. For kaolinite, a positive difference, i.e., greater strength response for the calcitic over the dolomitic lime, is shown for all lime levels and all curing treatments; the differential is magnified at elevated curing temperatures and high lime concentrations. For montmorillonite, a modest but clear-cut negative difference, i.e., dolomitic slightly more effective than calcitic, is indicated for curing at room temperature. But with elevated-temperature curing, there seems to be no clear-cut differences between the two limes.

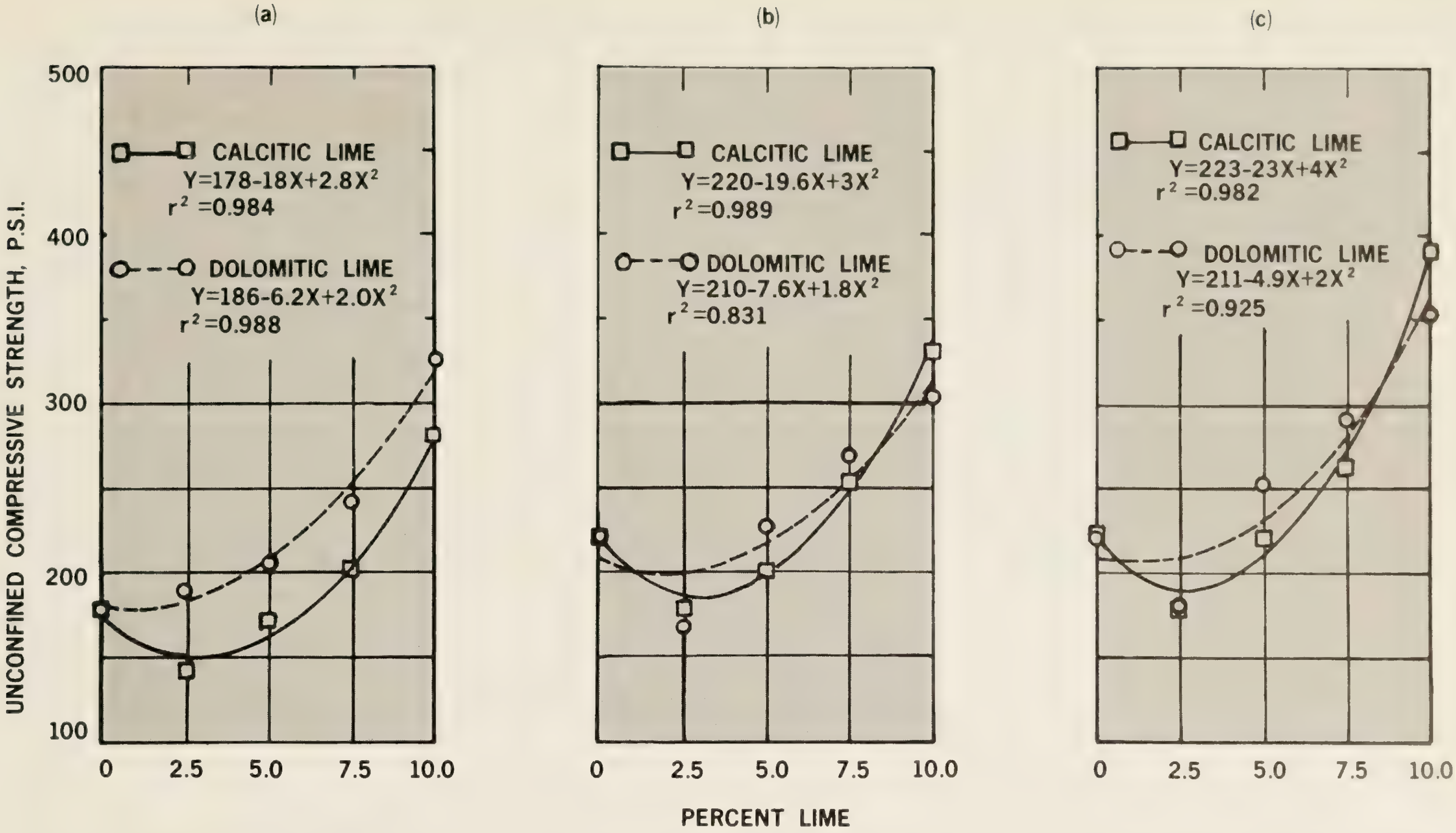

Figure 4.—Strength vs. lime concentration for montmorillonite-lime mixtures: (a) Specimens compacted immediately after mixture preparation, moist-cured 28 days at room temperature. (b) Specimens compacted immediately after mixture preparation, moist-cured 2 days at 120°F. (equivalent to about 45 days at 70°F.). (c) Loose mixture mellowed 48 hours at room temperature, specimens prepared and then moist-cured 2 days at 120°F.

Wang (*9*) suggested that the strength developed in soil-dolomitic lime mixtures was due to cementation resulting from the hydration of MgO to $Mg(OH)_2$; Andrews and O'Flaherty (*10*), in their explanation of the strength behavior of kaolinitic soil-lime mixtures, came to the conclusion that MgO acted as a catalyst in clay-$Ca(OH)_2$ reactions. The present study does not completely clarify the role of MgO. As indicated in the following paragraph, $Mg(OH)_2$ was not detected as a reaction product in cured mixtures; and in kaolinite-lime mixtures, a catalytic effect of MgO cannot be invoked because the dolomitic lime was less effective than the calcitic, and any benefit of the MgO would thus have been obscured in the overall greater strength values derived from the calcitic lime.

X-ray diffraction indications

X-ray diffraction patterns for the cured kaolinite-10 percent lime mixtures, figures 6 and 7, give no indication of the

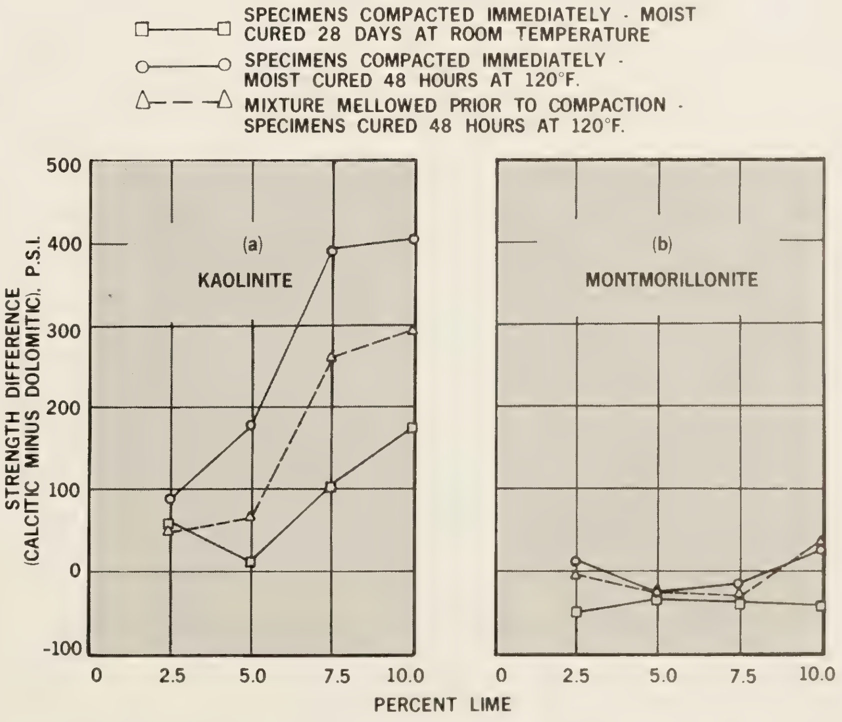

Figure 5.—Differences in strength response: Calcitic minus dolomitic lime.

formation of a new calcium silicate phase (diffraction peak at two theta approximately equal to 29.4° (3.03 Å)) or calcium aluminate phases (peaks at numerous positions). There is a slight suggestion of degradation of the kaolinite—as indicated by decreasing peak heights at two theta approximately equal to 12.5°, 23°, and 38°. Slight degradation was also noted for mixtures containing 5.0 and 7.5 percent lime and cured at 120°F. In figure 7, the absence of peaks at two theta approximately equal to 18.6° and 38° indicates that $Mg(OH)_2$ was not formed in the curing of the dolomitic lime mixtures. Unreacted MgO and $Ca(OH)_2$ (main peak of MgO at two theta approximately equal to 43° and main peaks for $Ca(OH)_2$ at two theta approximately equal to 18° and 34°) were found in some of the high-lime compositions.

The significant consumption of lime without appreciable degradation of kaolinite or the appearance of new definite diffraction peaks, indicates that the reaction products are X-ray amorphous and may be relatively small in quantity. A slight diffraction "activity" at two theta approximately equal to 6° (d = about 14 Å) was shifted by 350°C. heat treatment (pattern not included) to about 10 Å, indicating that the original kaolinite may contain a small amount of vermiculitic impurity. The peak was not shifted to lower angles when the material was solvated with glycerol, indicating that the impurity was not montmorillonite.

In the X-ray diffraction patterns for cured mixtures of montmorillonite and 10 percent calcitic lime, figure 8, there is evidence of some degradation of the montmorillonite (lessening intensities of the basal peaks at two theta approximately equal to 10, 15, 20, and 25°) and the formation of some CSH (gel) material (peak at two theta approximately equal to 29.4°, see curves c, d, and e). Unreacted $Ca(OH)_2$ (peak at 18° two theta) was not indicated for any of the cured specimens. X-ray diffraction patterns for montmorillonite mixtures containing 7.5 percent lime were similar to those for the compositions containing 10 percent lime.

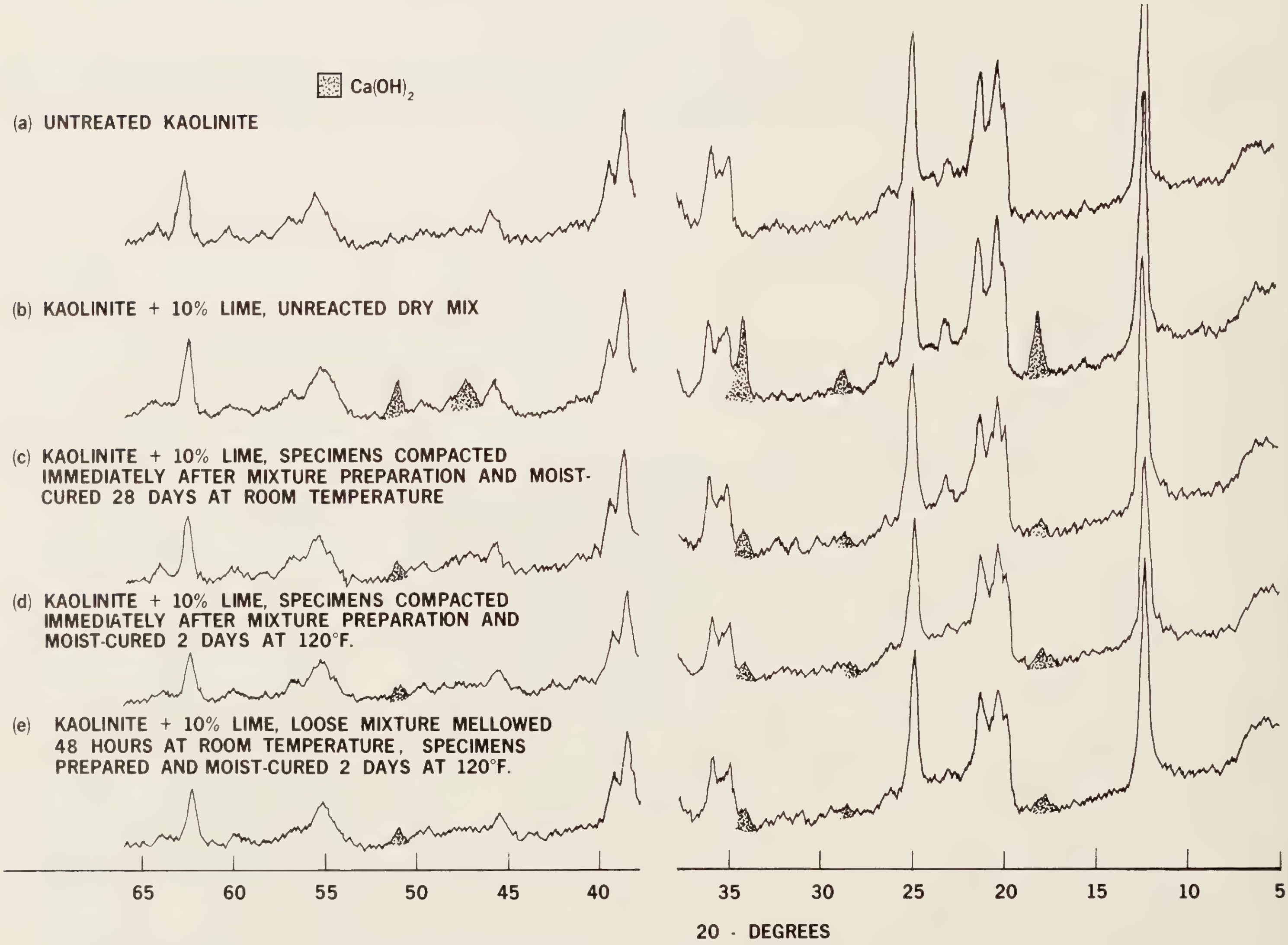

Figure 6.—X-ray diffractograms for kaolinite treated with calcitic lime.

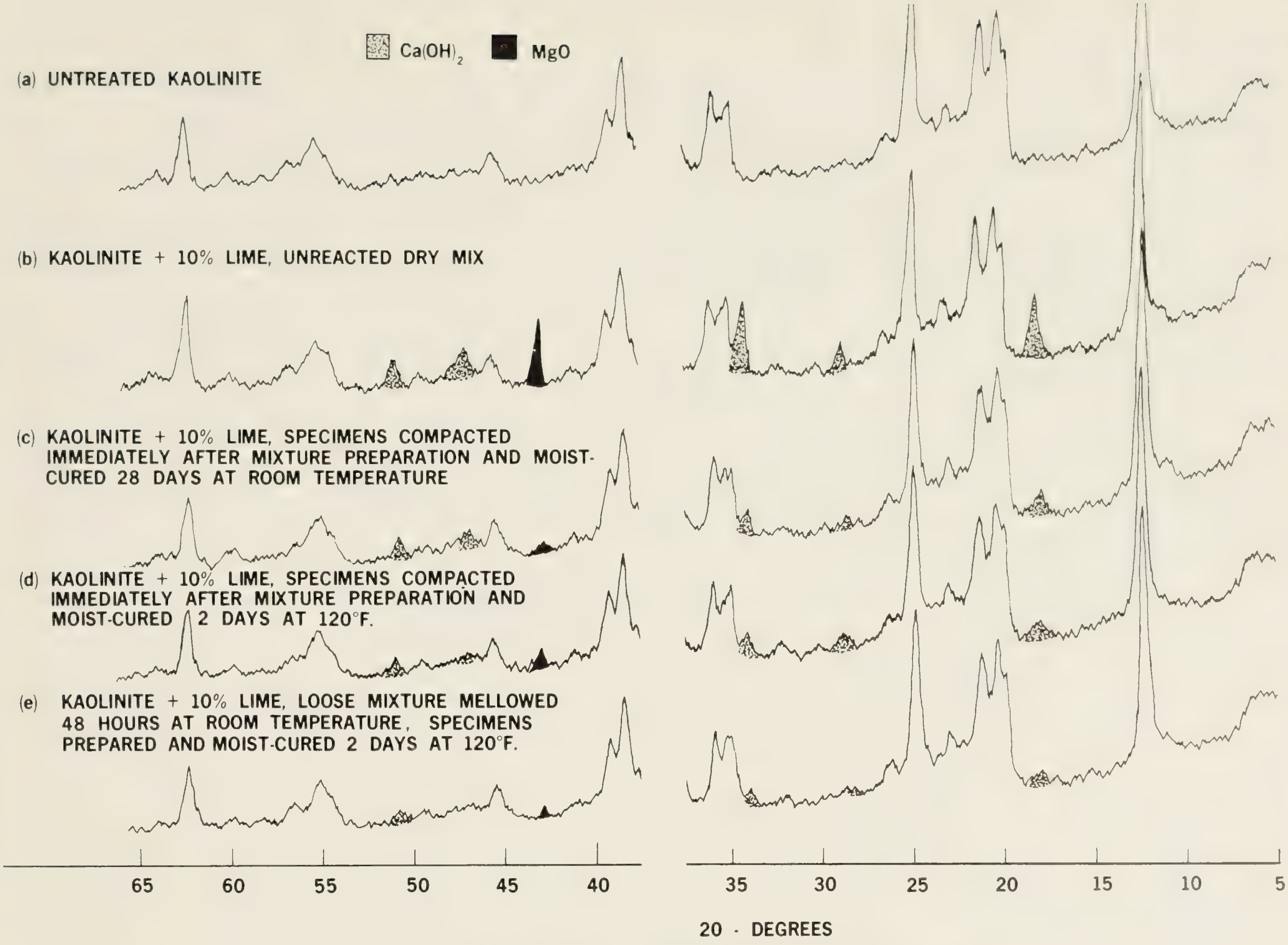

Figure 7.—X-ray diffractograms for kaolinite treated with dolomitic lime.

In patterns for 2.5 and 5 percent lime (not included) there was little evidence of clay degradation, but there was some indication of the formation of CSH (gel).

X-ray diffraction patterns for cured mixtures of montmorillonite and 10 percent dolomitic lime are given in figure 9. These patterns are practically identical to those for calcitic lime, indicating partial degradation of the montmorillonite and the formation of some CSH (gel) material. No $Mg(OH)_2$, Mg-containing silicates, or calcium aluminates are indicated. Lower percentages of dolomitic lime yielded X-ray diffraction results similar to those for corresponding percentages of calcitic lime.

The X-ray evidence for the presence of CSH (gel) in cured montmorillonite-calcitic lime mixtures, suggests that this product is of the same type as that synthesized by Kantro et al. (*20*), in studies relating to the hydration of portland cement, and found by Wang (*8*) in his studies of soil-lime stabilization. The CSH (gel) product is obviously very poorly crystallized (having only one diffraction peak).

Scanning electron microscopy

Scanning electron microscope pictures and elemental calcium distribution maps of fracture surfaces of untreated (calcium-saturated) and lime-treated kaolinite and montmorillonite specimens are given in figures 10 and 11. The lime-treated specimens contained 10 percent dolomitic monohydrated lime. These pictures are typical of a large number of fracture surfaces which were studied. No reaction product of recognizable morphology was found in these examinations—(c) of figures 10 and 11. The distribution of calcium was quite uniform in the kaolinite and montmorillonite samples—(d) of figures 10 and 11. The slight calcium activity indicated in figures 10(b) and 11 (b) is from the exchangeable calcium present as a result of calcium saturation.

Conclusions

Unconfined compressive strength determinations and X-ray diffraction studies were made of kaolinite-lime-water and montmorillonite-lime water mixtures with a calcitic and dolomitic lime, with curing of mixtures at room temeprature and at 120°F. Results of these studies are summarized as follows:

- Strength properties of the kaolinite-calcitic lime-water mixtures were superior to kaolinite-dolomitic lime-water mixtures. In mixtures with kaolinite the consumption of both types of lime was appreciable, but any products formed were not identifiable by X-ray diffraction.

- The strength properties of the montmorillonite-lime-water mixtures cured at elevated temperatures did not vary greatly with the type of lime. With curing at room temperature, the dolomitic lime produced slightly higher strengths than did the calcitic lime. The consumption of $Ca(OH)_2$ and MgO was appreciable in all of the montmorillonite systems, and X-ray identifiable CSH (gel) was formed.

- Evaluation of the strength properties of specimens cured 28 days at room temperature suggests that MgO does not function appreciably either as a catalyst for pozzolanic reactions or to bring about particle cementation by hydrating to $Mg(OH)_2$.

- The observation that cementitious effects occurred in the kaolinite systems without the formation of X-ray identifiable hydrated calcium silicates or aluminates suggests that stabilization may occur through processes involving the formation of very poorly organized gel material.

Recommendations

The scope of this study was limited to the comparison of the effects of calcitic vs. dolomitic limes (and, by implication, the role of magnesium) in the treatment of single representatives of two important clay mineral families. By use of the pure minerals it was hoped that some clarification of apparent contradictions in the results of earlier limited work with a few clayey soils would be provided and the feasibility and orientation of a more comprehensive and practical study with representative soil types would be indicated.

The results of the present study confirm that, in the treatment of basically different soil types, very substantial differences in effectiveness of the two lime types should be expected. This is indicated because (1) with the specific kao-

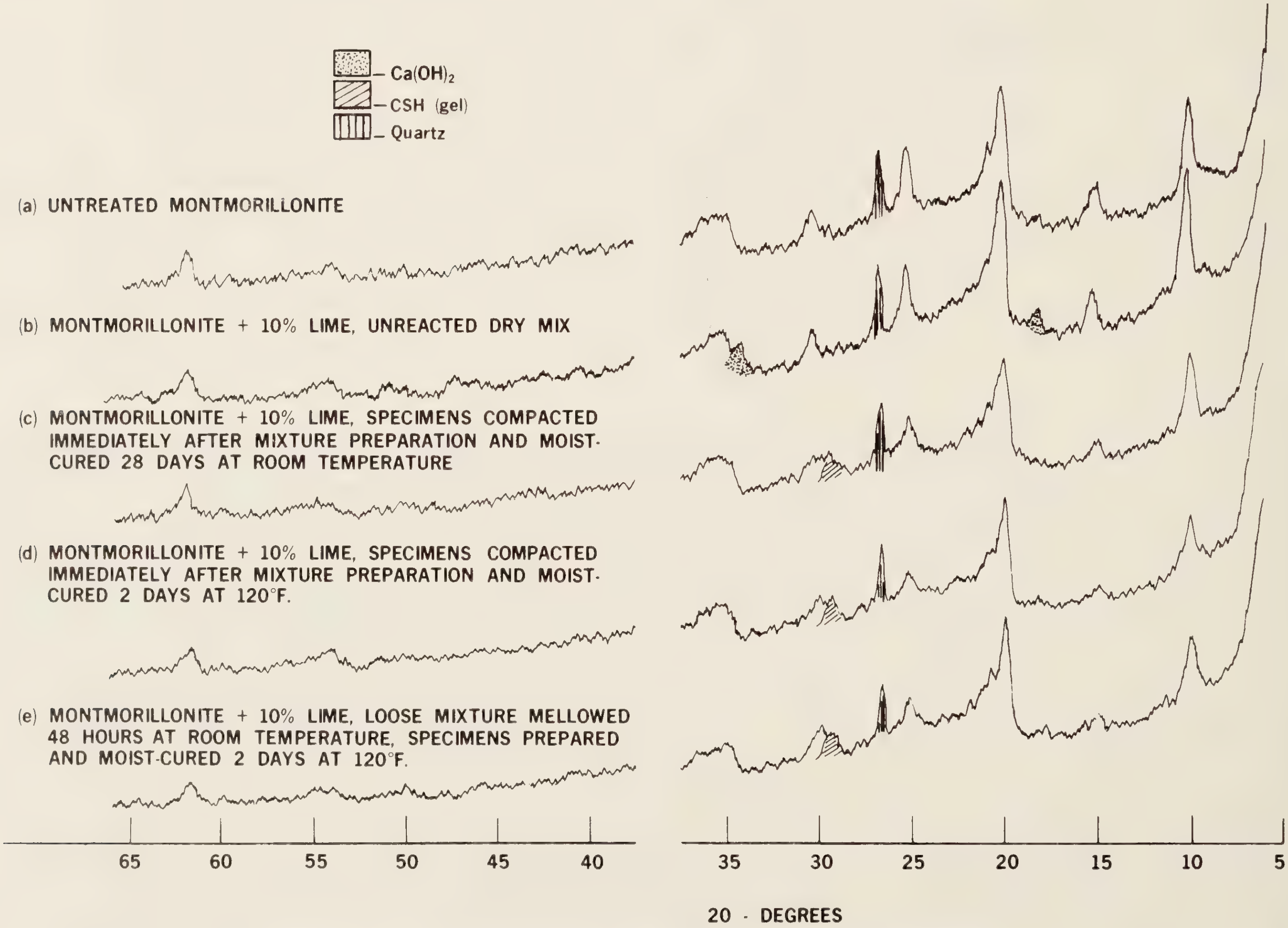

Figure 8.—X-ray diffractograms for montmorillonite treated with calcitic lime.

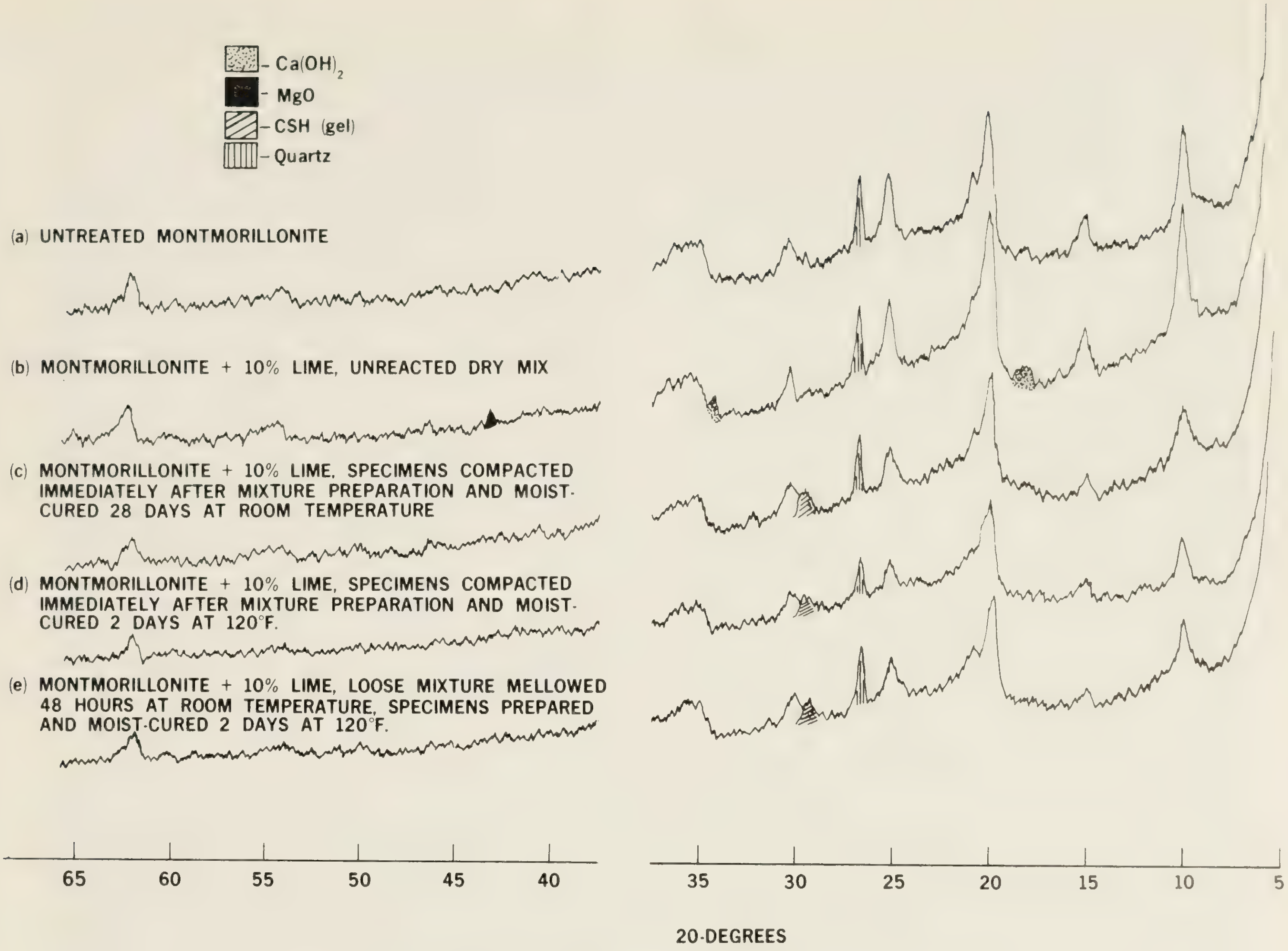

Figure 9.—X-ray diffractograms for montmorillonite treated with dolomitic lime.

linite studied, calcitic lime is considerably more effective in strength production, and (2) with the montmorillonite sample the dolomitic lime is slightly superior to the calcitic. It is apparent that the magnesium in the dolomitic lime plays a role that is not clearly understood.

At first glance, the results appear to indicate that calcitic lime should be selected for treatment of all kaolinitic soils, such as those in areas of southeastern United States; and that in parts of the west and midwest, where many soils are strongly montmorillonitic, either type of lime could be utilized without concern for the specific soil type. However, a number of factors complicate the lime selection process for any specific soil type or soil area so that such conclusions are not warranted by the limited data from the present study. In most soils the clay fraction is actually a mixture of clay minerals, and the dominant one from the lime-reactivity standpoint, is not clearly evident. Basic differences exist among different members of the kaolinite family as well as among different montmorillonite members and these most likely influence lime reactivity. In addition amorphous, lime-reactive, clay-size materials (such as hydrous aluminas, among others) are known to be present in most soils, often in substantial proportions and the relative reactivity of these with different lime types has not been measured. Consequently, continuation of research in this area is needed before the objective can be attained to develop practical recommendations for selecting the most effective type of lime for specific major U.S. soils.

The findings of the present investigation confirm that strength development in clay-lime mixtures is strongly affected by both lime type and type of clay mineral. Extensive survey, mapping and study of U.S. soils in matters such as soil type, parent material, clay mineral content, chemical characteristics, etc., have been performed and recorded. It is therefore recommended that the next phase of research in this area be a practical study of the effect of lime type on strength development and related properties of a group of soils selected so as to be representative of major types found in the United States.

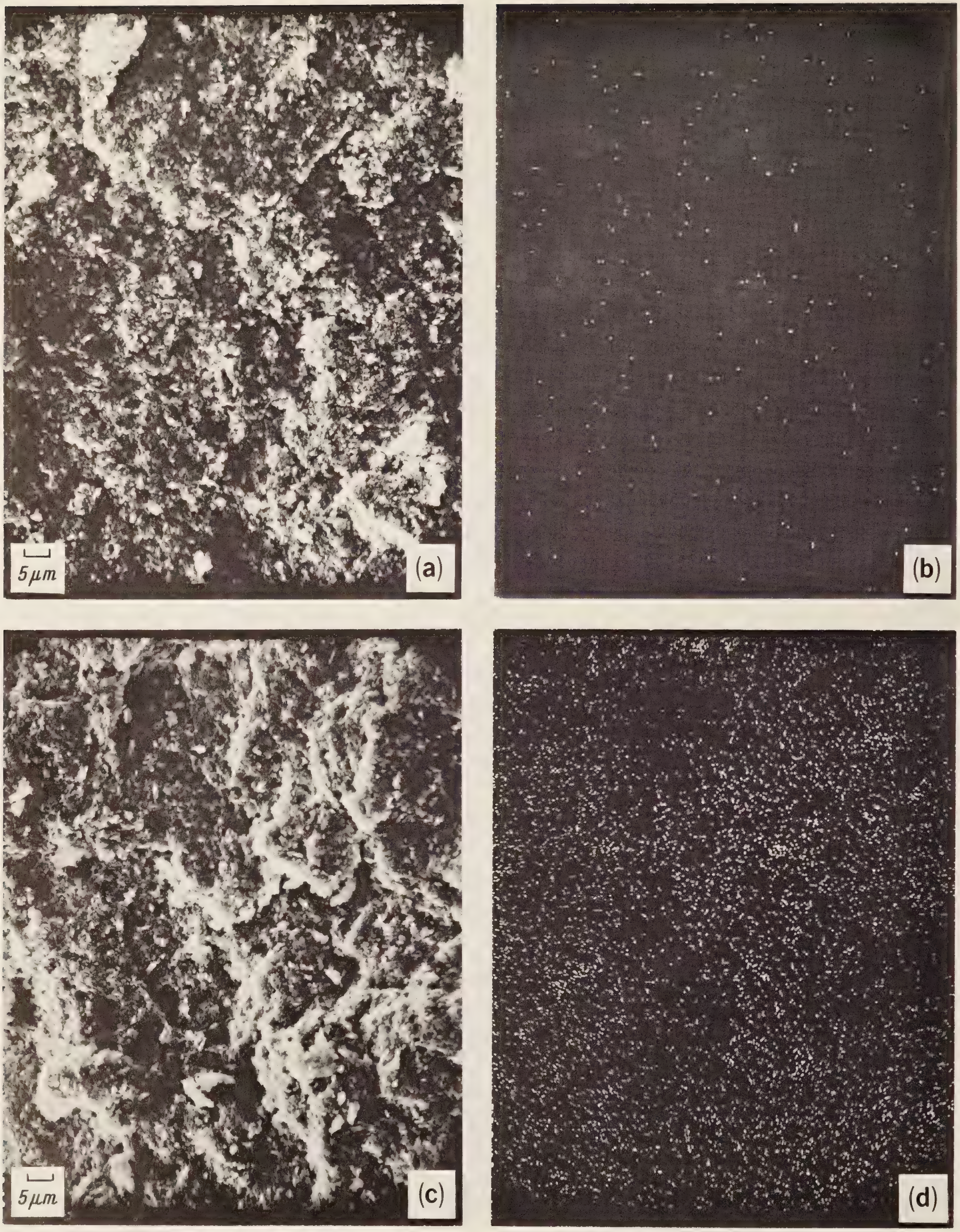

Figure 10.—Scanning electron micrographs and calcium element maps of untreated—(a) and (b)—and limt-treated—(c) and (d)—kaolinite cured 48 hours at 120°F.

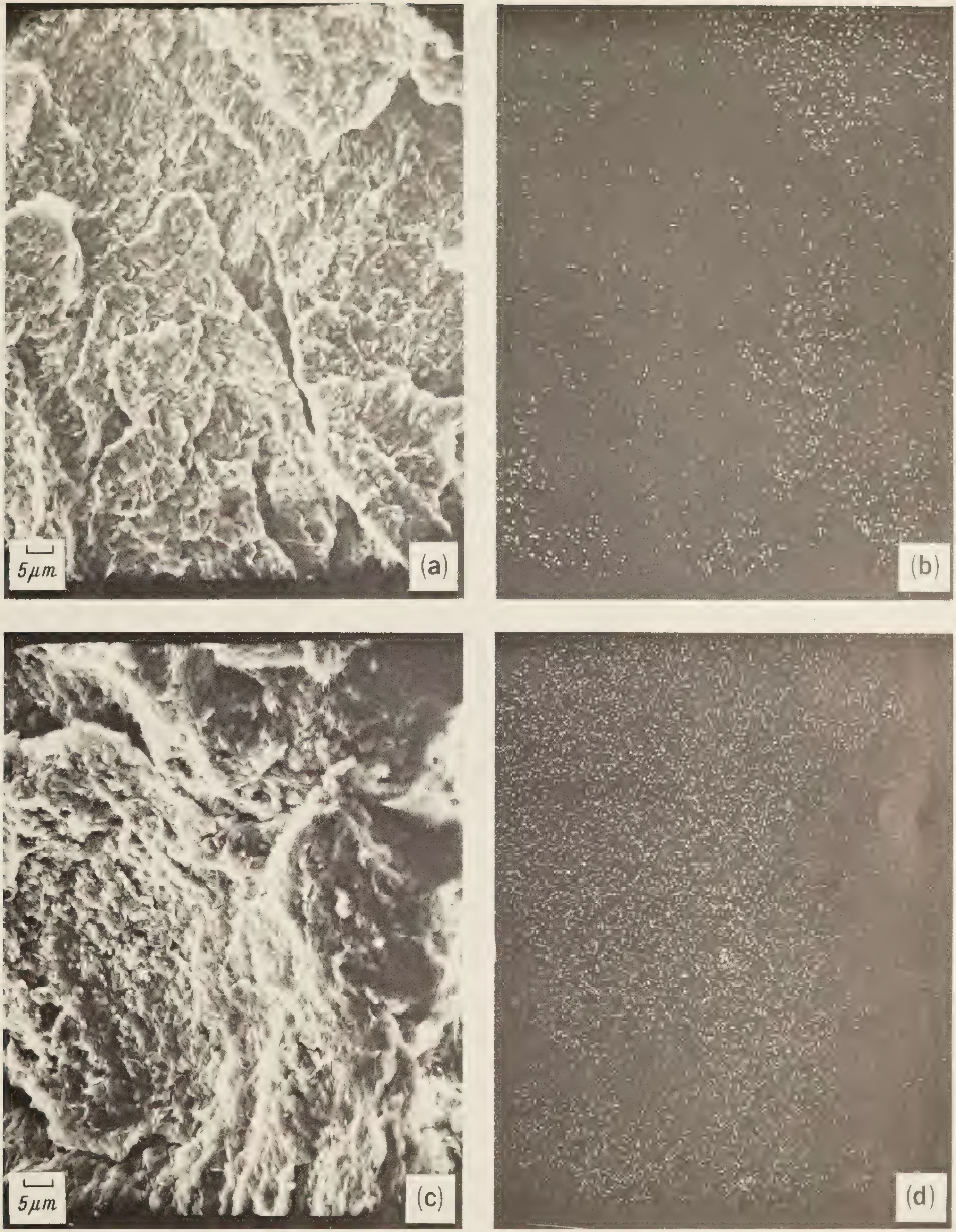

Figure 11.—Scanning electron micrographs and calcium element maps of untreated—(a) and (b)—and lime-treated—(c) and (d) —montmorillonite cured 48 hours at 120°F.

ACKNOWLEDGEMENTS

The authors express their thanks to E. F. Bailey, who assisted in all phases of the experimental work, and to Donald G. Fohs for valuable discussions.

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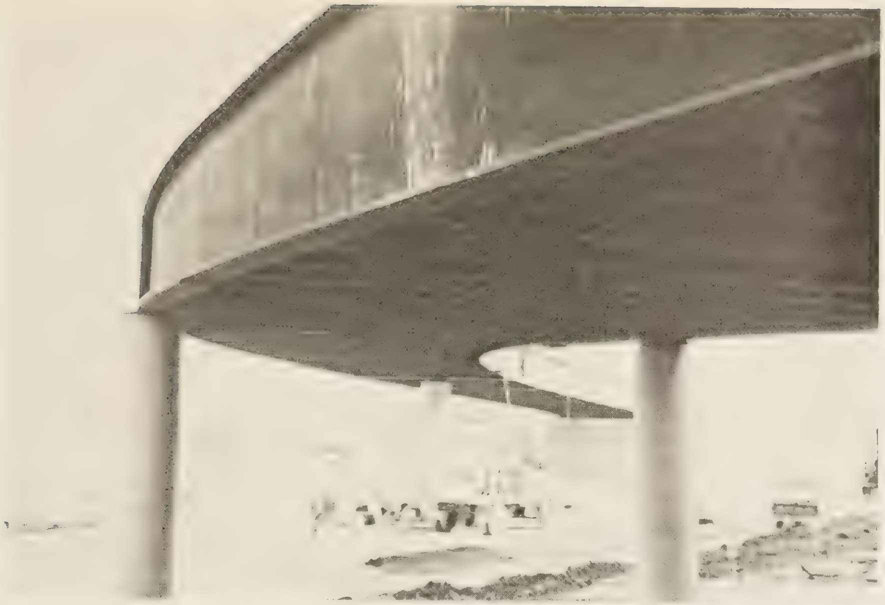

Design of Continuous Curved Bridges

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SPONSORED BY THE OFFICE OF RESEARCH

The design of continuous curved girder bridges offers to the bridge engineer a great challenge. Approximate techniques are available in estimating the forces in a curved bridge system. In order to examine the validity of these methods and implement the computer programs, which have been developed under the sponsorship of the Maryland State Highway Administration and the Federal Highway Administration (FHWA), a series of curved bridges, which are to be built in Maryland, were designed. In this article, the authors report the results of these bridge designs.

Introduction

DURING the past several years the Civil Engineering Department of the University of Maryland has been extensively engaged in research related to the design and behavior of curved girders. This research has resulted in various research reports and computer programs. Implementation of this research has been progressing in the following areas:

(1) Examination of the results for possible inclusion in a tentative AASHO code for curved bridges.

(2) Actual design of given bridge structures utilizing the research techniques.

The latter part of these two areas of implementation is discussed in detail in this article.

Techniques to establish the initial section properties and evaluate the final stresses are outlined as follows:

(1) Using a straight girder design computer program, the initially required flange sizes were established assuming a web size of 60 in. by 7/16 in. for A-588 steel.

(2) The girder flange areas were then increased by 30 percent to account for the torsional warping effects.

(3) An analysis was then performed using the U.S.S. (United States Steel Corp.) approximate technique to establish resulting moments due to

a. Direct loading.
b. Diaphragm action.
c. Flange bending.

(4) Stress resultants were then computed, indicating stresses slightly greater than the design stress of 27.0 k.s.i.

Using these resultant girder properties, as given in table 1, a curved girder computer oriented analysis was then conducted. Results of this study are described in the following paragraphs.

Design-Analysis Technique

A sophisticated analytical technique has been developed (*1, 2, 3*)[1] which incorporates interaction of all curved bridge elements. This technique has been formulated into a series of computer programs designated as COBRA (curved orthotropic bridge analysis) (*4*). The computer programs can accommodate single-, two- and three-span continuous curved bridges with radial supports. The programs also offer a major feature in that girder warp-

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[1] Italic numbers in parentheses identify the references listed on page 166.

Table 1.—Girder properties

Location	Properties									
	Noncomposite					Composite				
	K_T–in.^{4}	I_w–in.^{6}	I_x–in.^{4}	C_B–in.	W_n–in.^{2}	K_T–in.^{4}	I_w–in.^{6}	I_x–in.^{4}	C_B–in.	W^n_B–in.^{2}
A	56.	32.8×10^5	66,587.	28.4	366.	1,395.6	77.05×10^5	129,480.	43.8	755.
B	335.	62.4×10^5	137,815.	32.8	378.					
C	4.6	2.3×10^5	24,265.	28.7	214.	1,370.5	7.49×10^5	63,064.	50.1	435.
D	33.2	6.5×10^5	47,489.	31.5	216.					
E	4.6	2.2×10^5	24,265.	28.7	214.					

ing, designated as bimoment, is determined directly. This warping condition is especially important when studying the dead load (DL) effects.

At present, other computer programs are available for curved bridge design. However, these methods, generally a matrix GRID system, at present do not account for girder warping. The GRID technique does, however, account for the proper distribution of the remaining forces, as will be demonstrated.

The COBRA technique has also been corroborated through extensive testing of scale bridge models (*5, 6, 7*).

Description of Bridges

The two bridge structures that will be designed are two- and three-span continuous, respectively. The inside girder radius for both bridges is 430.5 feet. There are four girders, spaced at 8-foot intervals. The length of the girders and spacing of the diaphragms are shown in table 2.

The preliminary girder properties for the two bridges are listed in table 1. Bending stiffness properties were computed in the conventional manner. Torsional properties were determined by using a series of equations (*8, 9*), which were verified through laboratory testing (*10*). The location of the various sections are given in figure 1.

Bridge Loadings

The magnitude of the dead loads that were applied to the two structures were computed in the general manner. The magnitude of the live loads (LL) were in accordance with the AASHO code. The following load responses were examined in this study.

Two span

(1) Dead load, neglecting diaphragm action
(2) Dead load, including diaphragm action
(3) Negative moment effect—lane loading + 18
(4) Positive moment effect—AASHO truck

Three span

(1) Dead load, neglecting diaphragm action
(2) Negative moment effect—lane loading 2 spans + 18
(3) Positive moment effect—AASHO truck

It should be noted that the stiffening effects of the diaphragms, when the structure is subjected to only dead load, was also studied. This provided some insight as to the effects of the diaphragms in reducing the torsional influence.

Bridge Analysis

Comparisons between COBRA and GRID techniques

As described previously, there are various computer programs which can be utilized in the analysis and, eventually, design of curved girders. It is always reassuring to compare analytical results by two different methods. Therefore, for the analysis of the two-span structure when subjected to: (1) Dead load—no diaphragms; (2) dead load—with diaphragms; and (3) live load + dead load, the solution has been obtained by the COBRA and GRID computer techniques.

The results of these studies are given in tables 3 and 4. Generally, the bending moments (M), and deflections (d) agree favorably under dead loads. However, pure torsion (T) does not agree due to the exclusion of the bimoment (Bi) effect.

Table 2.—Girder dimensions

Structure	Dimensions							
	Girder spacing	Radius		Inside girder length			Diaphragm spacing	
		Inside	Outside	Span 1	Span 2	Span 3	Radial	Lateral
	Feet	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Spaces*	*Spaces*
Two span	8.0	430.5	454.5	149.57	149.57		10	20
Three span	8.0	430.5	454.5	79.63	85.60	55.06	[1] 7 [2] 4	None

[1] For 1st and 2d span.

[2] For 3d span.

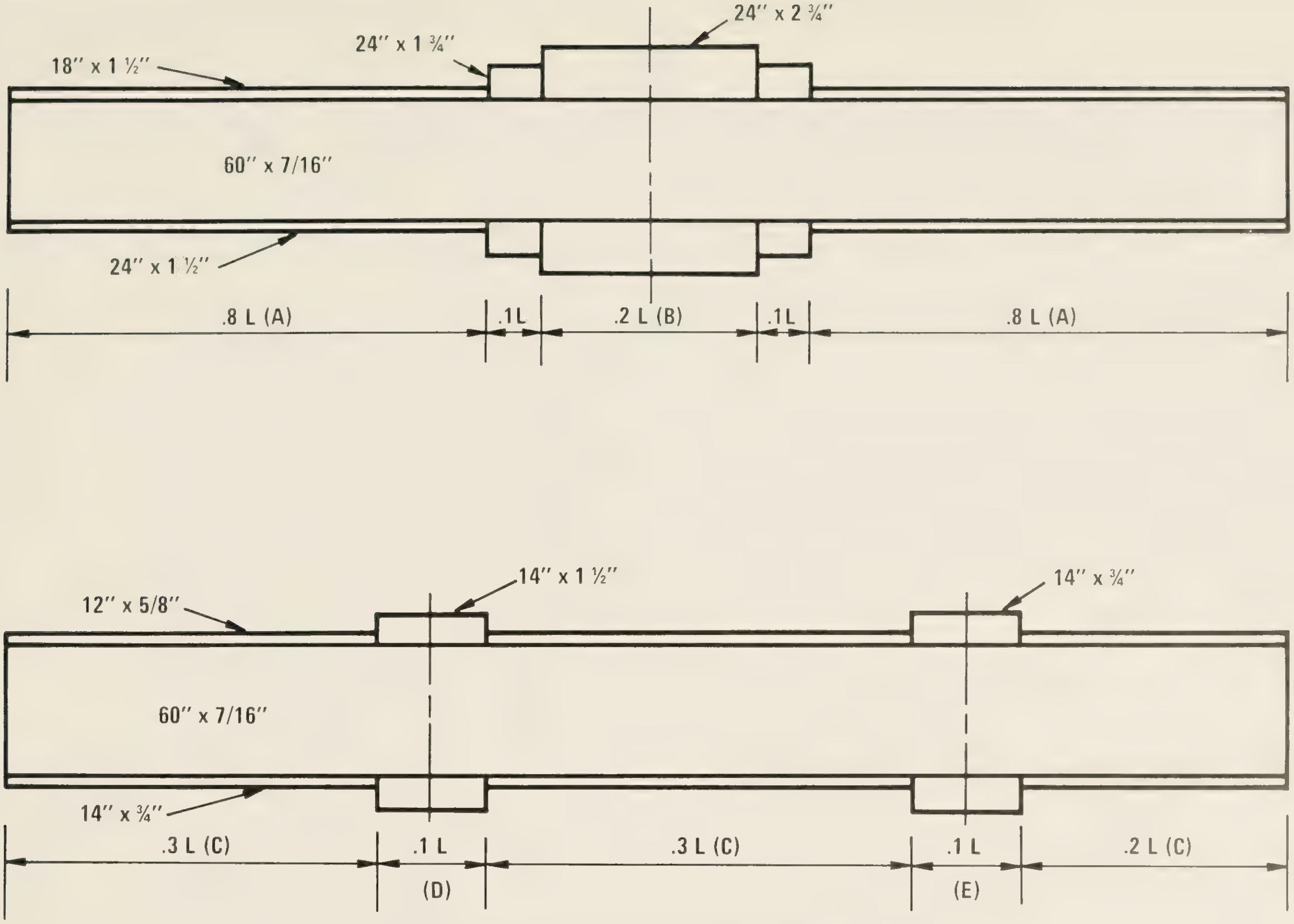

Figure 1.—Two- and three-span girder plans.

All of the live load effects, however, given by COBRA and GRID agree because the bimoment is not a predominant variable. This was demonstrated in laboratory tests (*5, 6, 11, 12*) and will be seen when the stress resultants are examined.

Force and deformation results

Utilizing the preliminary girder stiffness data (table 1), girder dimensions (table 2), and proper loads, the analysis of the bridges was performed. Maximum forces and vertical deformation in each bridge system under the various loads was then determined. These results were then plotted, and are described as follows:

(1) *Two-span structure*
   Dead load—neglecting diaphragms (fig. 2)
   Dead load—including diaphragms (fig. 3)
   Negative moment effect—lane loading (fig. 4)
   Positive moment effect—AASHO truck (fig. 5)

Table 3.—Two span data—COBRA

Function	[1] Girder No. 1	Girder No. 2	Girder No. 3	[2] Girder No. 4
Dead load—diaphragms included (noncomposite section)				
d—(in.)	3.44	6.75	7.26	3.63
T—(k-in.)	2,120.00	2,800.00	2,900.00	2,170.00
M—(k-in.)	38,200.00	62,100.00	64,500.00	25,100.00
V—(k)	94.40	170.80	183.80	88.60
Bi—(k-in.^{2})	16,900.00	46,500.00	47,100.00	23,800.00
Dead load—diaphragms not included (noncomposite section)				
d—(in.)	20.40	36.80	39.80	20.8
T—(k-in.)	754.00	1,230.00	1,290.00	675.00
M—(k-in.)	39,700.00	62,100.00	64,000.00	25,600.00
V—(k)	95.40	170.00	182.50	91.36
Bi—(k-in.^{2})	330,000.00	580,000.00	595,000.00	410,000.00
Live load—positive moment AASHO truck + dead load (composite section)				
T—(k-in.)	1.79	3.11	4.20	5.25
d—(in.)	388.00	178.00	276.00	280.00
M—(k-in.)	31,400.00	57,200.00	69,500.00	55,400.00
V—(k)	95.35	179.00	138.70	129.90
Bi—(k-in.^{2})	14,900.00	10,700.00	9,350.00	10,200.00

[1] Inside.
[2] Outside.

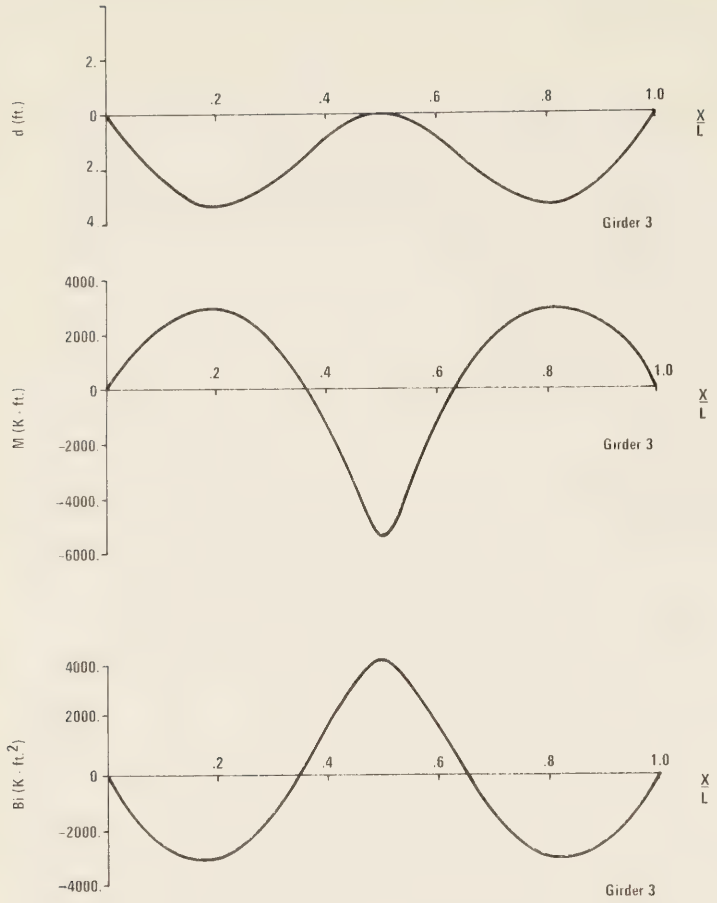

Figure 2.—Two-span structure (dead load effect—no diaphragms).

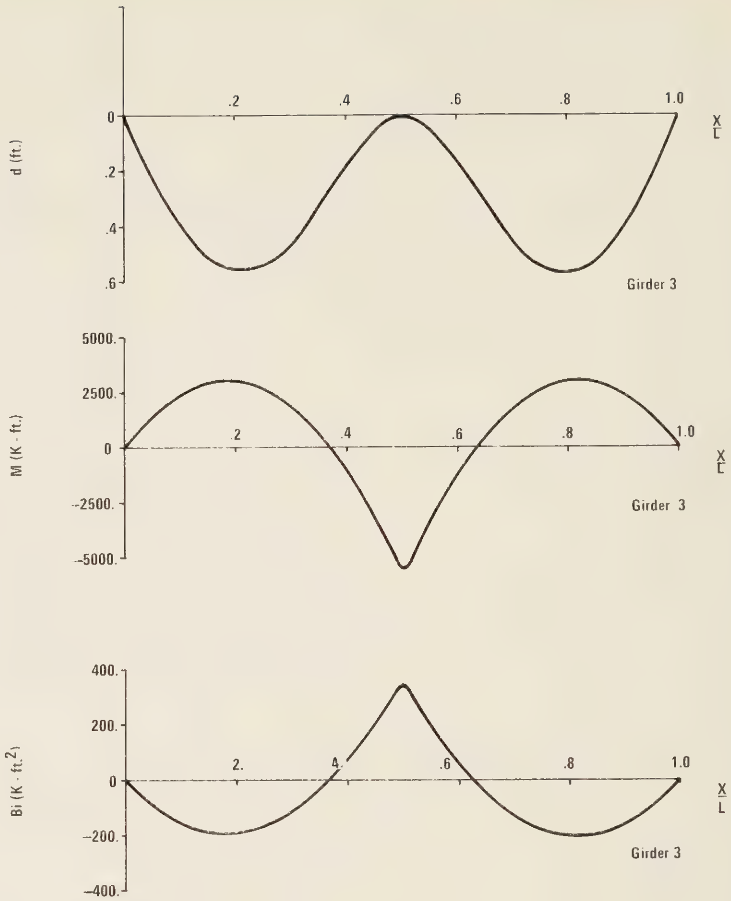

Figure 3.—Two-span structure (dead load effect—diaphragms included).

(2) *Three-span structure*

Dead load—neglecting diaphragms (fig. 6)

Negative moment effect—lane loading (fig. 7)

Positive moment effect—AASHO truck (fig. 8)

Resulting stresses

Using the resulting forces, as given in figures 2 through 8, and the girder properties (table 1), the resulting girder stresses were computed.

As described in table 5, the stresses on the bottom flange of the girder at midspan (section A) and support (section B) have been evaluated. The resulting maximum stress, excluding diaphragms, is 28.04 k.s.i., and including diaphragms is 26.27 k.s.i. Thus, the preliminary design section properties are adequate. It should be noted that the live load warping stresses are generally negligible. However, the dead load warping effects are 30 percent of the bending effects and thus not negligible.

Table 6 describes the resulting stresses on the three-span structure. The stresses are located on the bottom flange at midspan of Span 1 (section C), and the first interior support (section D). These results indicate a final stress of 27.9 k.s.i., which is acceptable, as the diaphragms would reduce the warping torsion effect.

Table 4.—Two span data-grid

Function	[1] Girder No. 1	Girder No. 2	Girder No. 3	[2] Girder No. 4
Dead load—diaphragms included (noncomposite section)				
d—(in.)	2.716	3.67	4.56	5.46
T—(K–in.)	14.8	13.3	11.96	12.2
M—(K–in.)	37,860.0	51,987.0	61,085.0	56,721.0
V—(K)	108.0	139.2	166.8	119.1
Dead load—diaphragms not included (noncomposite section)				
d—(in.)	38.14	80.19	84.71	44.89
T—(K–in.)	1,488.1	3,029.8	3,086.4	1,570.8
M—(K–in.)	34,701.6	70,645.9	71,959.0	36,625.4
V—(K)	88.12	176.15	176.19	88.1
Live load—positive moment AASHO Truck + dead load (composite section)				
d—(in.)	1.89	3.13	4.07	4.97
T—(K–in.)	360.2	361.9	284.2	306.2
M—(K–in.)	43,870.2	55,141.0	67,158.8	68,929.0
V—(K)	121.3	147.2	200.4	160.9

[1] Inside.

[2] Outside.

Table 5.—Two span analysis

[Steel stresses]

Loading	Bending moment K-in.	Bimoment K-in.^{2}	Bending stresses, k.s.i. stresses, k.s.i.—bottom	Warping stresses, k.s.i.—bottom	Maximum stress, k.s.i.—bottom
Dead load @ support (no bracing)[1]	66,000.	57,500.	15.70	3.48	19.18
Dead load @ midspan (no bracing)[2]	36,000.	46,000.	15.40	5.14 [3] 3.32	20.54 [3] 18.72
Negative moment lane loading @ support[1]	21,600.	4,900.	5.15	.210	5.36
Negative moment lane loading @ midspan[2]	12,000.	2,740.	4.06	.268	4.33
Positive moment AASHO truck @ support[1]	9,000.	2,740.	2.14	.167	2.31
Positive moment AASHO truck @ midspan[2]	17,400.	5,180.	5.90	.506	6.41

[1] Support is location B.

[2] Midspan is location A.

[3] Bracing included.

[4] $f_{max} = (I)(f_{LL}) + f_{DL} = 1.18 \times 6.4 + 20.54 = 28.04$ k.s.i.
$f_{min} = (I)\ (f_{LL}) + f_{DL} = 1.18 \times 6.4 + 18.72 = 26.27$ k.s.i.

Table 6.—Three span analysis

[Steel stresses]

Loading	Bending moment k-in.	Bimoment k-in.^{2}	Bending stresses, k.s.i. —bottom	Warping stresses, k.s.i. —bottom	Maximum stress, k.s.i.—bottom
Dead load @ support[1]	13,800.	14,400.	9.20	4.80	14.00
Dead load @ midspan[2]	10,200.	8,650.	12.00	8.10	20.10
Negative moment lane loading @ support[1]	5,750.	57.5	3.83	.019	3.85
Negative moment @ midspan[2]	4,320.	88.0	3.43	.050	3.48
Positive moment AASHO truck @ support[1]	4,800.	360.	3.20	.120	3.32
Positive moment AASHO truck @ midspan[3]	7,450.	650.	5.90	.38	6.28

[1] Support is location D.

[2] Midspan is location C.

[3] $f_{max} = (I)\ (f_{LL}) + (f_{DL}) = (1.24)\ (6.28) + 20.1 = 27.9$ k.s.i.

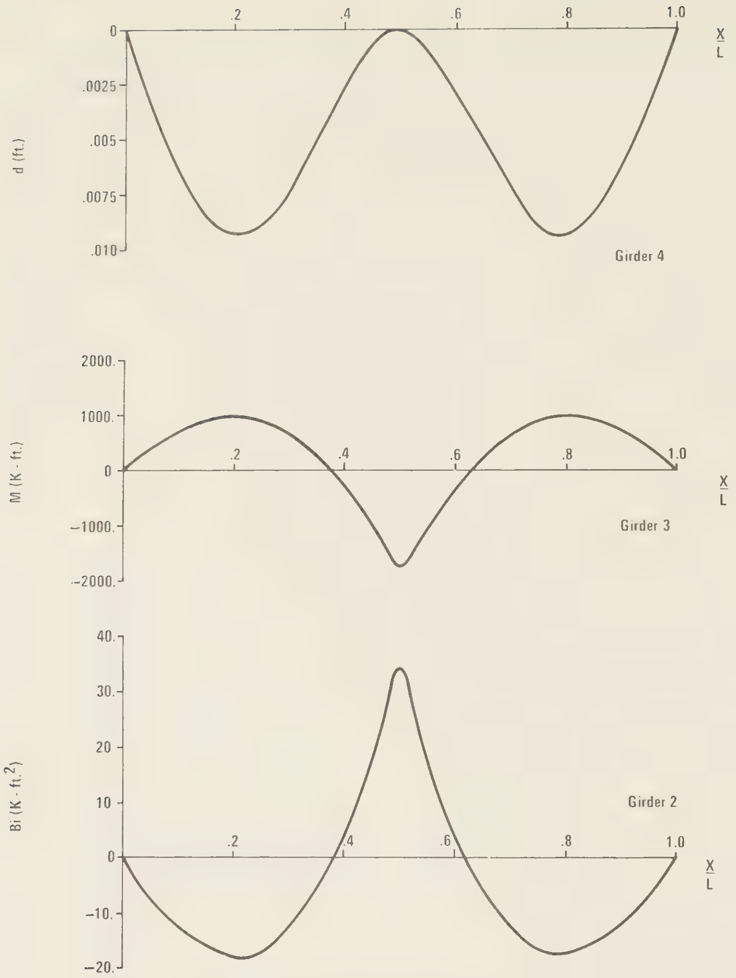

Figure 4.—Two-span structure (lane live load effect).

Results and Conclusions

The results of this study indicate that the preliminary design section properties are sufficient to withstand the dead and live load effects, and will not exceed design stresses.

The stress resultants demonstrate that the dead load warping condition is important and must be considered in design. Generally, the live load effect is negligible.

The stiffening effect of bracing can reduce the torsional effects, and thus reduce warping stresses. A design technique for estimating this influence should be examined in detail.

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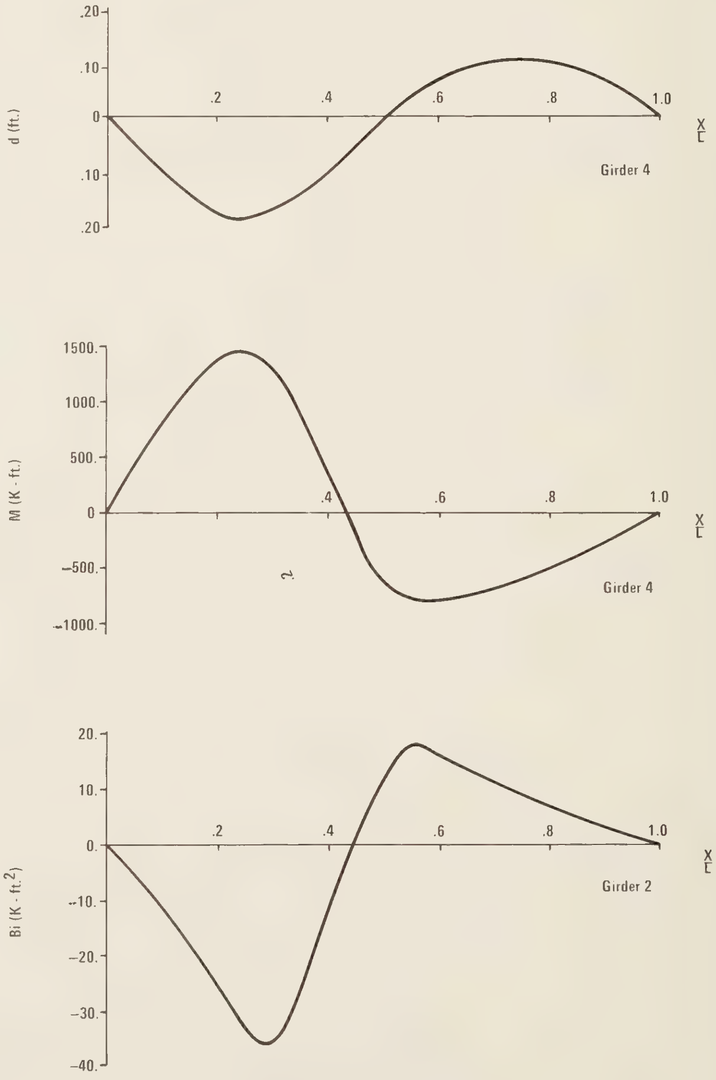

Figure 5.—Two-span structure (truck live load effect).

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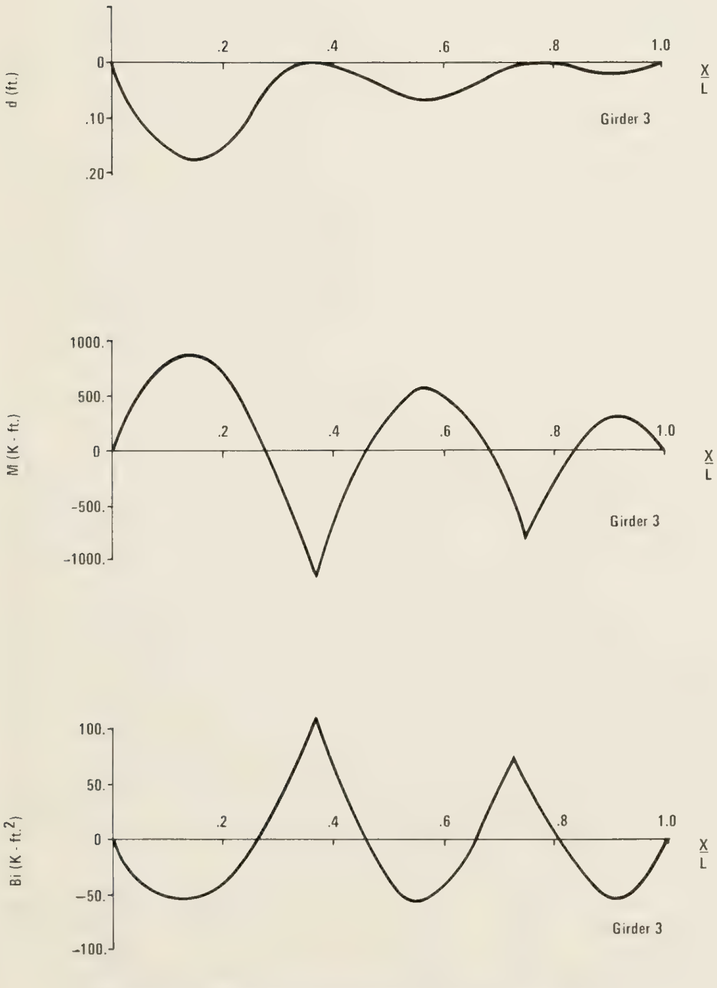

Figure 6.—Three-span structure (dead load effect).

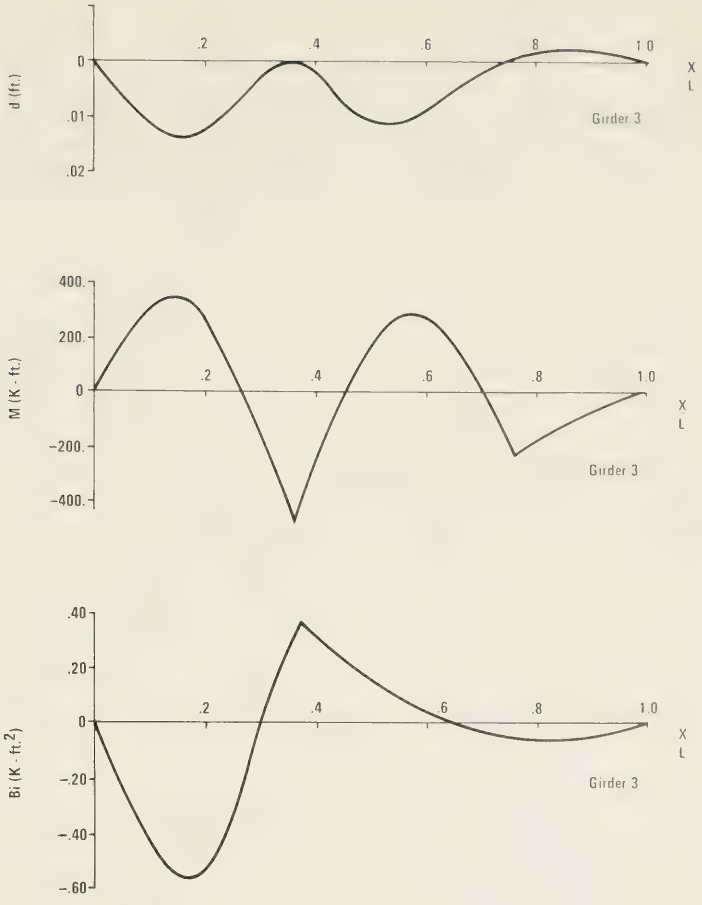

Figure 7.—Three-span structure (lane live load effect).

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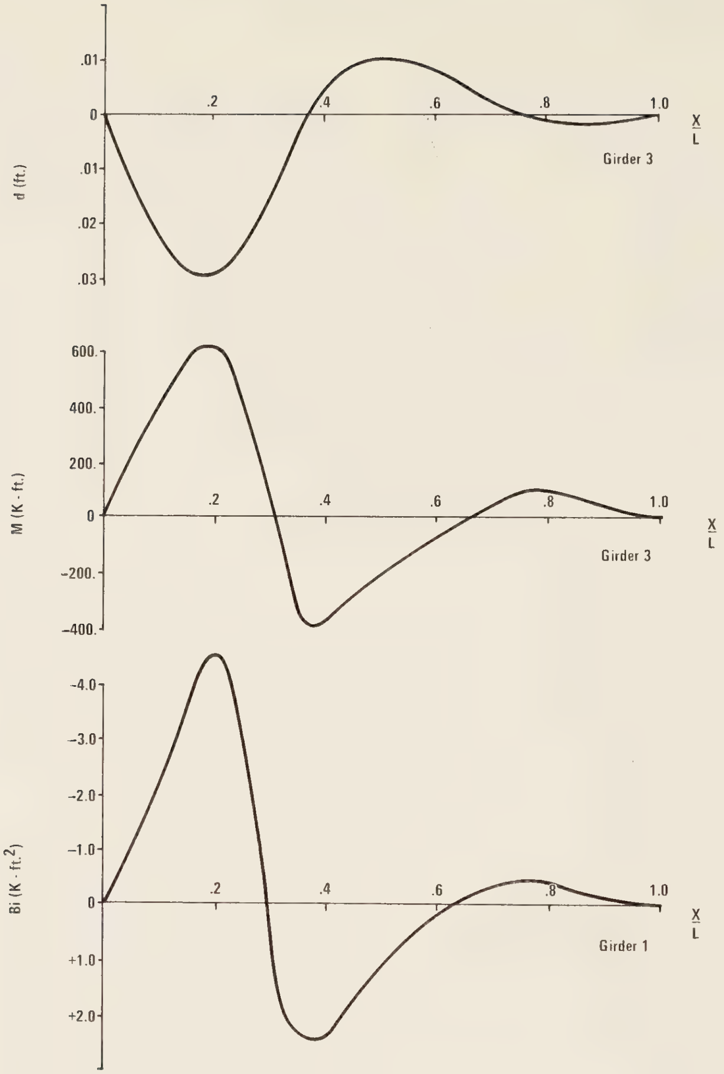

Figure 8.—Three-span structure (truck live load effect.)

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(*21*) J. Siminou, C. P. Heins, "Proposed Equations for Preliminary Design of Curved Girder Bridge Systems," *Civil Engineering* Report No. 25, June 1969.

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(*23*) H. Lee, C. P. Heins, "Local Loading Influence on Curved Decks," *Civil Engineering* Report No. 35, March 1970.

(*24*) B. Bonakdarpour, L. C. Bell, C. P. Heins, "The Behavior of a Curved Box Beam Model Bridge," *Civil Engineering* Report No. 36, June 1970.

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(*26*) Y. S. Ho, R. J. Reilly, "Analysis and Design of Curved Composite Box Girder Bridges," *Civil Engineering* Report No. 41, June 1971.

(*27*) D. L. Funkhouser, C. P. Heins, "Skew Supports Effects on Curved Bridges," *Civil Engineering* Report No. 44, February 1972.

New Research in Progress

The following items identify new research studies that have been reported by FHWA's Offices of Research and Development. These studies are sponsored in whole or in part with Federal highway funds. For further details, please contact the following: Staff and Contract Research—Editor; Highway Planning and Research (HP&R Research)—Performing State Highway Department; National Cooperative Highway Research Program (NCHRP)—Program Director, National Cooperative Highway Research Program, Highway Research Board, 2101 Constitution Avenue, N.W., Washington, D.C. 20418.

FCP Category 1—Improved Highway Design and Operations for Safety

FCP Project 1A: Traffic Engineering Improvements for Safety.

Title: Traffic Signal Warrants. (FCP No. 51A1504)

Objective: Evaluation of the adequacy of existing and additional warrants in determining whether a traffic signal should be installed. Research includes consideration of delays, volumes, capacity, accidents, comfort and convenience, and physical conditions to determine proper criteria for warrants including testing and verification of warrant values.

Performing Organization: KLD Associates, Inc., Huntington, N.Y. 11743

Expected Completion Date: September 1974.

Estimated Cost: $200,000 (NCHRP)

Title: Design Warrants for Left-Turning Vehicles at Signalized Intersections. (FCP No. 41A2512)

Objective: Develop a methodology for field measurement of delays caused by left-turning vehicles, quantify from field measurements the effect of left turns on capacity and/or delay, and develop usable and practical methods for determining the type of provision required for left-turning vehicles at signalized intersections.

Performing Organization: Department of Transportation, Trenton, N.J. 08625

Expected Completion Date: February 1974.

Estimated Cost: $64,000 (HP&R)

FCP Project 1H: Skid Accident Reduction.

Title: Friction Characteristics of Paving Materials in Connecticut. (FCP No. 41H1174)

Objective: To relate the performance of various types of pavement materials and surfaces under differing conditions of traffic to reduction in friction level. To investigate and to define indicators on the hydroplaning potential of pavement surfaces, and to define seasonal variations in skid numbers which many roads show.

Performing Organization: Department of Transportation, Hartford, Conn. 06115

Expected Completion Date: August 1974.

Estimated Cost: $26,000 (HP&R)

Title: Improve Portland Cement Concrete Wearing Surfaces. (FCP No. 41H1184)

Objective: To determine through field experimentation and observation the effects that construction techniques and materials have on the production of durable textured concrete surfaces which result in enduring skid resistance. To recommend changes in contract controls and construction procedures accordingly.

Performing Organization: Division of Highways, Sacramento, Calif. 95807

Expected Completion Date: August 1977.

Estimated Cost: $155,000 (HP&R)

Title: Field Test and Evaluation Center for Western States. (FCP No. 31H8253)

Objective: Establish, maintain and operate a field test and evaluation center for standardizing the measurement of pavement skid resistance. Provide technical assistance and calibration services to State highway departments and other agencies in Western States.

Performing Organization: Ford Motor Company, Yucca, Ariz. 86438

Expected Completion Date: December 1974.

Estimated Cost: $442,000 (FHWA Administrative Contract)

Title: Field Test and Evaluation Center for Central and Southern States. (FCP No. 31H8283)

Objective: Maintain and operate a field test and evaluation center for standardizing the measurement of pavement skid resistance. Provide technical assistance and calibration services to State Highway departments and other agencies in Central and Southern States.

Performing Organization: Texas Transportation Inst., College Station, Tex. 77843

Expected Completion Date: December 1973

Estimated Cost: $367,000 (FHWA Administrative Contract)

FCP Project 1J: Improved Geometric Design.

Title: Study of Collector-Distributor Roads. (FCP No. 41J2042)

Objective: To determine when collector-distributor roads should be used; and develop design standards where their use is justified.

Performing Organization: Purdue University, Lafayette, Ind. 47907

Expected Completion Date: June 1974.

Estimated Cost: $30,000 (HP&R)

Title: Speed of Vehicles on Grades. (FCP No. 41J3282)

Objective: This study will obtain new field data concerning motor vehicle operating characteristics on selected grades and relate these data to current and future geometric design standards, capacity and safety.

Performing Organization: University of Texas, Austin, Tex. 78701

Expected Completion Date: August 1974.

Estimated Cost: $69,000 (HP&R)

FCP Project 1L: Improving Traffic Operations in Substandard Visual Environments.

Title: Evaluation of Illumination Designs for Accident Reduction at High Nighttime-Accident Highway Sites. (FCP No. 41L2102)

Objective: To develop a reliable, objective technique, utilizing information seeking eye-movement patterns of test drivers. For highway officials to quickly evaluate the potential accident-reduction effectiveness of illumination design additions at known high nighttime-accident sites.

Performing Organization: Ohio State University, Columbus, Ohio 43210

Expected Completion Date: October 1974.

Estimated Cost: $58,000 (HP&R)

Title: Speed Advisory Information for Reduced Visibility Conditions. (FCP No. 31L4032)

Objective: To develop preliminary design specifications for a speed advisory system which will inform drivers of the current speed of traffic on the road ahead during periods of adverse weather when visibility is reduced.

Performing Organization: Department of Transportation, Salem, Ore. 97310

Expected Completion Date: December 1974.

Estimated Cost: $263,000 (FHWA Administrative Contract)

FPC Project 1O: Aids to Surveillance and Control.

Title: Development of Techniques to Evaluate New and Existing Railroad Passive Protection Devices. (FCP No. *41O1023)*

Objective: The study will develop: (1) New passive protection devices (signs, crossbucks designating the location of railroad tracks) and (2) Techniques to evaluate passive protection devices. The study will evaluate new and existing passive protection devices.

Performing Organization: Department of Transportation, Trenton, N.J. 08625

Expected Completion Date: December 1974.

Estimated Cost: $39,000 (HP&R)

FCP Category 2—Reduction of Traffic Congestion and Improved Operational Efficiency

FCP Project 2B: Development and Testing of Advanced Control Strategies in the Urban Traffic Control System.

Title: Signal Progression on Routes in Suburban Areas. (FCP No. 42B2291)

Objective: To develop criteria for the introduction of signal progression on major State-controlled signalized arterials in suburban areas. Also, to determine maximum signal spacing for which progression is beneficial and determine the effect of flow interruption from secondary roads between signalized intersections.

Performing Organization: Ohio State University, Columbus, Ohio 43210

Expected Completion Date: November 1974.

Estimated Cost: $77,000 (HP&R)

FCP Category 3—Environmental Considerations in Highway Design, Location, Construction and Operation

FCP Project 3F: Pollution Reduction and Visual Enhancement.

Title: Superior Bluegrass for Roadside Turf. (FCP No. 43F2482)

Objective: Test promising varieties of bluegrass under actual highway conditions and develop suitable means to produce seed necessary for the tests.

Performing Organization: Purdue University, Lafayette, Ind. 46207

Expected Completion Date: October 1977.

Estimated Cost: $65,000 (HP&R)

Title: Low Cost Maintenance Program for Indiana Roadsides. (FCP No. 43F2492)

Objective: In order to improve vegetation and maintenance practices this study will test data from previously completed research.

Performing Organization: Purdue University, Lafayette, Ind. 46207

Expected Completion Date: October 1978.

Estimated Cost: $75,000 (HP&R)

FCP Category 4—Improved Materials Utilization and Durability

FCP Project 4C: Use of Waste as Material for Highways.

Title: Synthetic Aggregate from Incinerator Residue by a Continuous Fusion Process. (FCP No. 34C1013)

Objective: Establish feasibility of converting municipal incinerator residue to synthetic aggregate for highway use by a continuous fusion process; prepare preliminary design for fullscale production plant.

Performing Organization: Franklin Institute, Philadelphia, Pa. 19103

Expected Completion Date: October 1973.

Estimated Cost: $56,000 (FHWA Administrative Contract)

FCP Category 5—Improved Design to Reduce Costs, Extend Life Expectancy and Insure Structural Safety

FCP Project 5D: Structural Rehabilitation of Pavement Systems.

Title: Environmental Deterioration of Pavement. (FCP No. 45D2022)

Objective: To provide usable and economical methods of improving remedial maintenance and pavement design and reducing costs of construction and maintenance by (A) identifying the causes of pavement cracking in West Texas and (B) testing the efficiency of selected remedies for the problem.

Performing Organization: Texas Transportation Institute, College Station, Tex. 77843

Expected Completion Date: August 1974

Estimated Cost: $40,000 (HP&R)

FCP Project 5G: Predicting the Service Life of Bridges.

Title: Stress History of Highway Bridges. (FCP No. 45G1242)

Objective: A study of the stress history of two steel girder bridges with welded cover plates and one prestressed concrete bridge when subjected to large volumes of heavy truck traffic.

Performing Organization: University of Tennessee, Knoxville, Tenn. 37916

Expected Completion Date: December 1974

Estimated Cost: $83,000 (HP&R)

Title: Design Parameters for Columns in Bridge Bents. (FCP No. 45G3672)

Objective: To simplify and improve the reliability of the design processes for columns in highway bridges.

Performing Organization: University of Texas, Austin, Tex. 78712

Expected Completion Date: August 1974.

Estimated Cost: $64,000 (HP&R)

Title: Box and I-Curved Bridges (FCP No. 45G3682)

Objective: To develop techniques for determining requirements for external bracing in curved I-Girder bridges, internal diaphragm spacing and stiffness for curved box beam bridges, influence lines for preliminary design and to field test a curved box beam bridge.

Performing Organization: University of Maryland, College Park, Md. 20742

Expected Completion Date: June 1976.

Estimated Cost: $125,000 (HP&R)

Title: Bridge Vibration Studies. (FCP No. 45G4072)

Objective: To establish design specifications which will directly regulate dynamic response characteristics of steel girder bridges.

Performing Organization: Purdue University, Lafayette, Ind. 47907

Expected Completion Date: June 1977.

Estimated Cost: $170,000 (HP&R)

FCP Project 5J: Rigid Pavement Systems Design.

Title: Improved Pavement-Shoulder Joint Design (FCP No. 55J1252)

Objective: Study the performance of the longitudinal joint between PCC roadway and bituminous surfaced shoulder and relate behavior to climate, support, pavement/shoulder design, traffic, drainage and sealing materials/techniques. Develop and evaluate improved systems for maintaining a seal and minimizing infiltration. Prepare a field study program for highway agencies.

Performing Organization: Georgia Institute of Technology, Atlanta, Ga. 30332

Expected Completion Date: September 1974.

Estimated Cost: $100,000 (NCHRP)

FCP Category 6—Efficiency in Construction and Maintenance Operations

FCP Project 6D: Improved Methods, Materials and Equipment for Highway Construction.

Title: Bearing Capacity of Piles from Dynamic Measurements. (FCP No. 46D4103)

Objective: Complete the development, field testing and implementation of the case pile capacity computer.

Performing Organization: Case Western Reserve University, Cleveland, Ohio 44106

Expected Completion Date: November 1974.

Estimated Cost: $95,000 (HP&R)

FCP Category 7—Implementation of Research and Development Findings

FCP Project 7A: Develop and Apply New Techniques and Methods to Highway Engineering.

Title: Automated Design of Prestressed Concrete Beams Made Continuous for Live Loads. (FCP No. 47A3143)

Objective: Study existing automated design procedures. Develop design functions to be performed and program format. Determine proper method of analysis for questionable items such as ultimate moment capacity at interior supports, shear capacity, and effects of creep, shrinkage, and temperature. Develop and assemble program subroutines and test our system.

Performing Organization: Texas Transportation Institute, College Station, Tex. 77840

Expected Completion Date: August 1974.

Estimated Cost: $44,000 (HP&R)

Non-FCP Category 0—Other New Studies

Title: Center Barrier Visibility. (Non-FCP No. 40E3224)

Objective: Effects of illumination on center barrier visibility and locatability from driver point of view; characteristics of sample delineation devices and treatments; effectiveness and practicality of sample installations acceptability of promising techniques.

Performing Organization: Department of Transportation, Trenton, N.J. 08625

Expected Completion Date: December 1974.

Estimated Cost: $101,000 (HP&R)

Title: Lightweight Aggregate in Asphalt Concrete. (Non-FCP No. 40M1323)

Objective: To determine asphalt-absorption characteristics of lightweight aggregates and utilize them to develop a mix design procedure, based on the Marshall method, for asphalt concrete mixtures containing lightweight aggregates.

Performing Organization: Department of Transportation, Albany, N.Y. 12226

Expected Completion Date: May 1974.

Estimated Cost: $72,000 (HPR)

Title: Structural Evaluation of Asphalt-Aggregate Cold Mix Bases. (Non-FCP No. 40M1354)

Objective: To establish a method for evaluating the pertinent properties of bases constructed with granular materials stabilized with emulsions or cutbacks and to develop procedures for utilizing this information in pavement design.

Performing Organization: University of Illinois, Urbana, Ill. 61801

Expected Completion Date: July 1976.

Estimated Cost: $171,000 (HP&R)

Title: Corrosion of Steel Piles. (Non-FCP No. 40M3142)

Objective: Identify parameters, such as soil conditions and construction methods, that may have an effect on steel pile corrosion.

Performing Organization: Division of Highways, Sacramento, Calif. 95807

Expected Completion Date: June 1975.

Estimated Cost: $247,000 (HP&R)

Highway Research and Development Reports Available From the National Technical Information Service

The following highway research and development reports are available from the National Technical Information Service, Sills Building, 5285 Port Royal Road, Springfield, Va. 22151. Paper copies are priced at $3 each and microfiche copies at 95 cents each or as otherwise indicated. To order, send the stock number of each report desired and a check or money order to the National Technical Information Service. Prepayment is required.

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PB 212024	Field Study of the Cost and Performance of a Precast Prestressed Composite Pavement on U.S. Highway 14 Bypass—Second Interim Report. (First Interim Report PB 200665).
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PB 212053	Induced Trench Method of Culvert Installation. (Illinois IHR-77—Final Report).
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PB 212838	The Economic Impact of 1-80 on Southwestern Wyoming: Summary and Conclusion.
PB 212839	Joints Fastened by Huckbolts. (Washington State Highway Commission/U.S. Department of Transportation, FHWA).
PB 212849	Field Testing of Horizontally Curved Steel Girder Bridges: Third Interim Report. (New York State Department of Transportation).
PB 213145	An Investigation of the Behavior of a Three-Span Composite Highway Bridge. (Missouri State Highway Department/U.S. Department of Transportation, FHWA).
PB 213149	Strip Footing Analysis. (U.S. Department of Transportation, Federal Highway Administration).
PB 213155	Maryland's Two Experimental Continuously Reinforced Concrete Pavement Projects—I-83, Baltimore-Harrisburg Expressway.
PB 213209	Loading History—Span No. 10 Yellow Mill Pond Bridge, I-95, Bridgeport, Connecitcut.
PB 213396	Analysis of Bending Stiffness Variation at Cracks in Continuous Pavements. (University of Texas Research Report 56-22).
PB 213397	Verification of Computer Simulation Methods for Slab and Girder Bridge Systems. (University of Texas Research Report 115-1F).
PB 213512	Moisture Content of Bridge Decks—Part 2.
PB 213552	Experimental Bituminous Concrete Pavements—I-84, Southbury-Middlebury, I-95, Groton, Final Report. (Connecticut Department of Transportation/U.S. Department of Transportation, FHWA).
PB 213567	Investigation of Field Prestress Grouting Procedures, Interim Report. (California Division of Highways/U.S. Department of Transportation, FHWA).
PB 213568	Method of Controlling Grout for Prestressed, Post-tensioned Structures. Interim Report. (California Division of Highways/U.S. Department of Transportation, FHWA).
PB 213696	Correlation of Seismic Velocities with Earthwork Factors. (California Division of Highways/U.S. Department of Transportation, FHWA).
PB 213726	Instability of Horizontally Curved Members—Bending Behaviors of Cylindrical Web Panels. Interim Report. (Pennsylvania Department of Transportation/U.S. Department of Transportation, FHWA).
PB 213736	A Sensitivity Analysis of Flexible Pavement System FPS2. Interim Report. (Texas Highway Department/U.S. Department of Transportation, FHWA).
PB 213962	Bridge Expansion Joint Seals. Final Report. (California Division of Highways Report No. CA-HY-BD-4105-3-72-06).

MATERIALS

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PB 212032	Basic Properties of Pavement Components. (FHWA-RD-72-19).
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PB 213170	Use of Hydrated Lime in Bituminous Mixtures to Decrease Hardness of the Asphalt Cement. Final Report. (Utah State Department of Highways Report No. 500-909).
PB 213398	Strength Development and Reaction Products in Lime-Montmorillonite-Water Systems. Interim Report. (Federal Highway Administration Report No. FHWA-RD-72-31).
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PB 213033	Pilot Study of Housetrailer and Truck Camper Safety, Phase II. Final Report. (University of California, B-1-44). Paper copy $6, microfiche 95 cents.
PB 213115	Railroad-Highway Safety Report to Congress, Part II—Recommendations for Resolving the Problem. (HNG-14/RA-30).